A JOURNAL OF HIGHWAY RESEARCH AND DEVELOPMENT

U.S. Department of Transportation Federal Highway Administration

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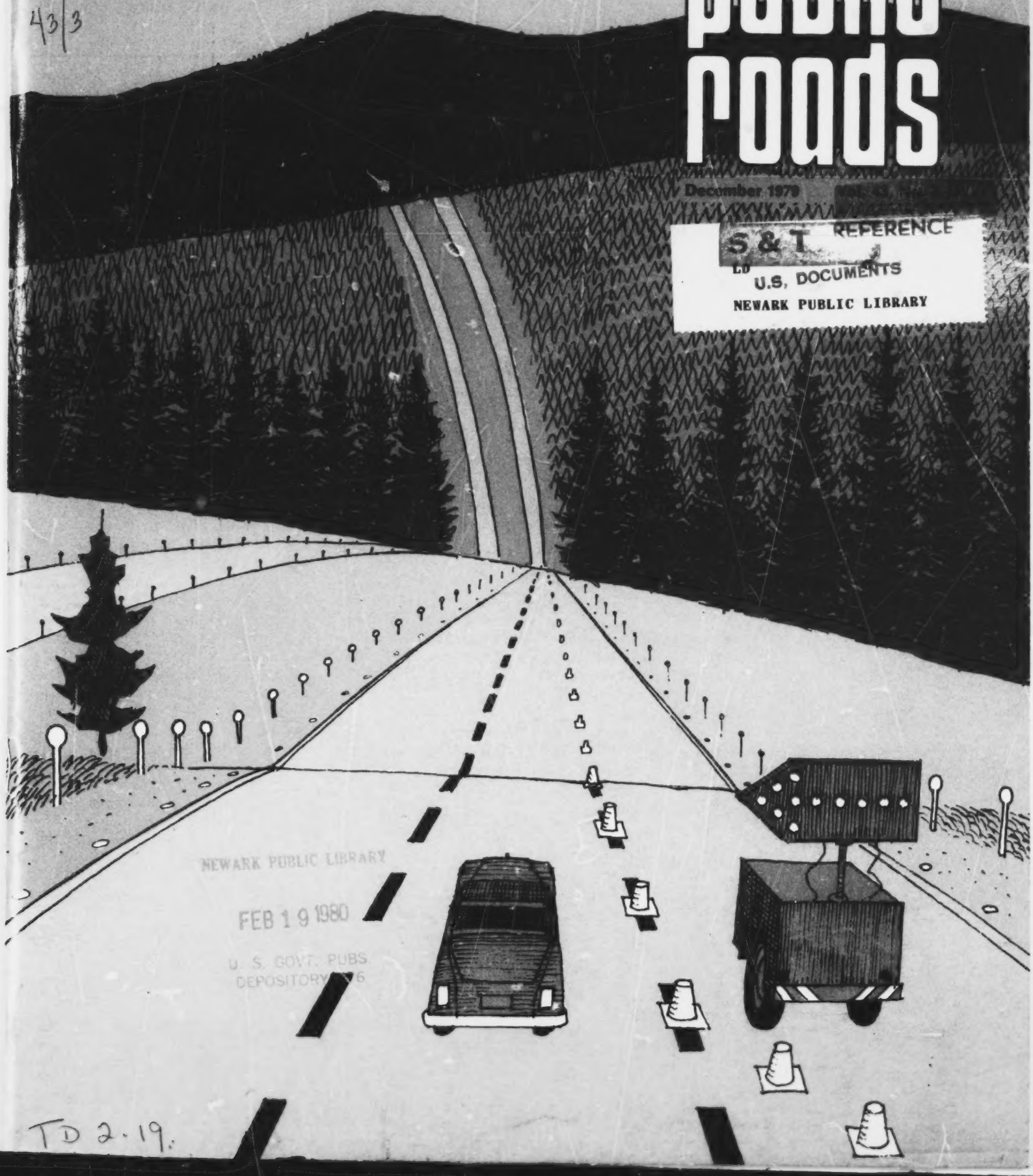

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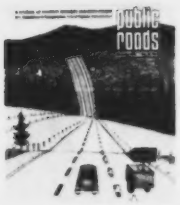

COVER:
Artist's concept of lane delineation techniques.

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Neil Goldschmidt, *Secretary*

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Karl S. Bowers, *Administrator*

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Is Delineation Needed?

by
Richard N. Schwab and Donald G. Capelle

The 1973 Federal-Aid Highway Act established a pavement marking demonstration program and a special research and development program to improve the effectiveness of highway delineation techniques. (*1*)[1] A major objective of the research program was to insure that delineation systems address motorists' needs for guidance and advisory information. This article discusses where, when, and how much delineation is required for safe and effective driving. Research that dealt with developing improved materials and technology for delineation was discussed in a previous article in *Public Roads*. (*2*)

What is Delineation?

Delineation is a device or treatment, excluding signs, that provides guidance and regulatory or warning information to a driver. (*3*) Delineation can be almost any feature of the roadway or its environment that shows the motorist the proper path to follow. These features can be devices specifically for delineation (painted centerlines or edgelines or postmounted retroreflectors installed on the shoulders) or construction details built into the roadway (joints in a portland cement concrete pavement, guardrails, rumble strips, or contrasting shoulder treatments [fig. 1]). Delineation also can be unintentional, such as an incidental part of the roadway environment—a fenceline, a row of trees, or the topography of the region (fig. 2). These surroundings, although visible in daylight, may not always provide sufficient nighttime guidance. Adequate guidance must be provided, especially at locations where natural or construction details may be misleading—for example, a pavement joint or a row of trees that continues straight ahead although the roadway curves.

Current U.S. roadway delineation practices are governed by the Manual on Uniform Traffic Control Devices (MUTCD). (*4*) The MUTCD provisions are general and permit various delineation treatments, materials, and patterns to compensate for different climatic and traffic conditions.

A series of research studies began in early 1975 to examine the effect of several types and patterns of delineation treatments on accident experience and traffic behavior. The goal of the research was to establish minimum and optimum delineation requirements based on driver behavior and to develop more specific recommendations for effective delineation.

Research Approach

A human factors study investigated photometric standards for lane markings and developed more visible yellow paint markings. (*5, 6*) This research involved a driving simulator that used computer-generated outlines of a road and markings projected on a screen in front of a real vehicle passenger compartment. The consequences of decreased marking visibility were evaluated by the driver's ability to control the vehicle and by subjective ratings. Instrumented vehicle studies under real-world traffic conditions were used to validate the simulator findings. Color identification of yellow highway paint under typical daytime and nighttime illumination conditions was also studied in both laboratory and field tests.

[1] Italic numbers in parentheses identify references on page 96.

Figure 1.—Asphalt pavement with contrasting shoulder treatment.

Figure 2.—Trees on both sides of a roadway provide natural delineation.

A traffic performance study measured the effects of delineation on vehicular control and accident potential. (*7*) Traffic performance measures, such as the speed and lateral placement of vehicles within their lane of travel, were evaluated in relation to delineation changes, geometric changes, and environmental changes.

An accident analysis study investigated accident, geometric, climatic, and traffic data collected at approximately 500 sites in 10 States. (*8*) Over 13,000 accidents were analyzed at two types of highway sites: Matching-control sites and before-and-after sites. At the matching-control sites, accidents occurring at two geometrically similar sites (with similar traffic and climatic conditions) with different delineation treatments were compared. The delineation treatments *at each site* remained the same over the analysis period. At the before-and-after sites, accident data were available both before and after the installation of a specific delineation treatment. Cost-benefit analyses were performed at sites where significant differences in accident frequency could be attributed to delineation treatment.

Driver comprehension of road markings was investigated in a questionnaire study that assessed a driver's understanding of marking system principles and his or her ability to interpret actual road situations shown on film. (*9*)

Findings and Recommendations

The major findings of the research studies described above will be discussed in relation to their effects on the three major delineation treatments—stripes, raised pavement markers, and postmounted delineators. Application of each

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treatment is also discussed. The results of previous studies are included.

Stripes

The most common type of delineation is the paint stripe. One of the first issues addressed in the research program was the need for striping. Although the need for pavement marking, especially for a centerline, is rarely questioned, there are many kilometres of low-volume, two-lane roadways without any marking. Table 1 gives mean accident rates for two-lane roadways with average daily traffic (ADT) volumes of 500 to 5,000 vehicles. (*8*)

Striping reduced accidents approximately 30 percent; the data were significant at the 0.05 level. If this finding is extrapolated to traffic volumes lower than those observed in the study, centerlines can be cost beneficial at ADT volumes as low as 50 vehicles.

Driver behavior studies have shown that adding a centerline to a previously unmarked roadway reduced the road's predicted hazard level by almost 50 percent. (*10*) Therefore, there is strong evidence that the centerline should be used whenever a roadway has a paved surface that will retain a paint marking and is wide enough to carry two-way traffic.

Edgeline striping, shown in figure 3, is a generally accepted practice on major roadways, although its effectiveness has been questioned. The human factors study showed improved lateral stability with more conspicuous edgelines. (*5*) Similar results were obtained in the traffic performance study where performance improved with edgelines. (*7*) Results of the accident analysis study showed that edgelines improved safety somewhat; however, this improvement was greater on straight roads than on winding roads. (*8*) This is contrary to what might be expected. The authors suggest that this may illustrate the role of stress on driver attentiveness; that is, a driver is less attentive on straight roads and appears to rely on edgelines if it is necessary to respond to a particular driving situation. On winding roads where a driver is under stress and therefore already attentive to the driving task, he or

Figure 3.—Roadway with painted edgelines.

Table 1.—Accidents per million vehicle-kilometres for two-lane roadways with ADT volumes of 500 to 5,000 vehicles

Delineation treatment	Type of site		
	Tangent	Winding	All
No treatment	2.4[1]	2.0	2.1
Painted centerline only	1.4	1.6	1.4

[1] Ten or fewer sites with relatively large variance.

1 veh-km=0.62 veh-mile

she will not rely as much on the edgelines for guidance. A recent 2-year accident study in England has shown similar results. (*11*) There is also evidence of a major accident reduction at intersections having edgelines.

It can be concluded that edgelines are important in a roadway delineation system and should be used on major roads wider than 6 m (20 ft). If traffic safety is the only consideration, an ADT volume of 1,000 vehicles is necessary to make edgelines cost beneficial. If other factors are considered, such as reduced costs for shoulder maintenance, edgelines may be justified on roads having ADT volumes lower than 1,000 vehicles.

Delineation is coded to convey certain information to drivers. Rules for coding, provided in the MUTCD, include the elements of color, shape, size, and configuration. (*4*) Use of a two-color system for pavement markings has been questioned in recent years. Yellow paint is slightly more expensive than white paint and is less visible under adverse weather conditions. The toxicity of the lead chromate pigment used in yellow paint is another concern. According to the MUTCD, a yellow pavement marking means that "traffic on the other side of a yellow marking" is moving in the opposite direction. (*4*) The questionnaire study demonstrated that many drivers do not understand this. (*9*) Most drivers associate yellow with warning signs and caution signals, that is, a hazard. Therefore, a more aggressive driver education program is needed for this code to be effective.

The human factors study also showed that the yellow paint presently used can be desaturated considerably in rural areas where there is little or no fixed roadway lighting. (*6*) A mixture of 50 percent white pigment to 50 percent yellow pigment, by weight, will not decrease color identification for most drivers. In urban areas having considerable fixed lighting, however, the dilution should be much less (not more than 30 percent white pigment, by weight, should be used), particularly as the use of sodium vapor light sources increases. Limited field evaluation of "lightened" yellow is underway in several areas of the United States and more research is planned. If the dilution of yellow pigment is operationally acceptable, the annual savings can be significant.

The MUTCD requires a stripe-to-gap ratio of 1:3 for both centerlines and lanelines. (*4*) Usually, stripes are 3.05 m (10 ft) long and gaps are 9.14 m (30 ft) long. The traffic performance study found little difference in driver performance for various stripe-to-gap ratios in dry weather when sight distance was good. (*7*) In wet weather the results were similar, although data were limited and the amount of rain was not quantified. The human factors study showed that in heavy rain the stripe-to-gap ratio has a significant effect on steering performance, especially when forward visibility is restricted. (*5*)

In summary, although the 1:3 stripe-to-gap ratio is adequate for most roadways, it may need to be supplemented in situations where forward visibility may be restricted (fig. 4). (*12*) In mountainous terrain or where climatic conditions commonly cause restricted visibility, the 1:3 stripe-to-gap ratio should be supplemented by raised pavement markers. Such markers show up particularly well at night in fog and rain when paint markings are not as effective.

Figure 4.—Loss of effectiveness of paint markings because of rain.

Raised pavement markers

Raised pavement marker use has greatly increased in the past 10 years, and markers now play an important role in many delineation treatments. A raised pavement marker is shown in figure 5. The major advantages of these markers over paint stripes are reduced maintenance and more positive all-weather, nighttime delineation. The major disadvantages are high initial cost and possible incompatibility with snowplows. Snowplowable raised pavement markers have become available in recent years but are more expensive. Early research with snowplowable markers indicated that they could not be used with tungsten carbide snowplow blade inserts. However, recent field experience with 1.5 million markers in Ohio indicated that this restriction may no longer be true.

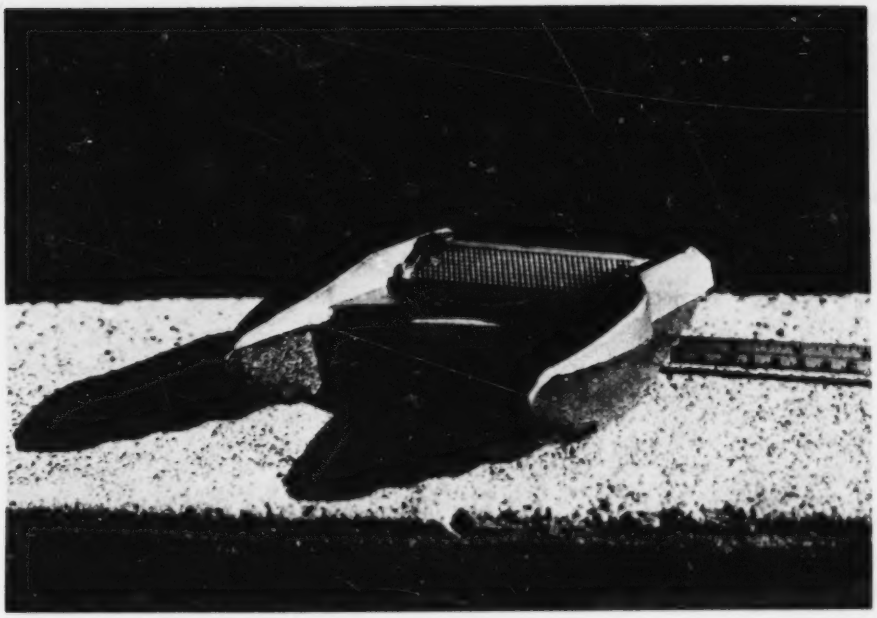

Figure 5.—A raised pavement marker before installation.

Raised pavement markers have widely replaced painted centerlines, especially in the sunbelt States (fig. 6). Typically, four nonreflective raised markers are used in place of the stripe, and a retroreflective marker is placed at the center of every other gap. This type of marking reduces the amount of lane changing and discourages encroachments onto opposing lanes, possibly because of the rumble effect produced when running over the markers. Research has shown that raised pavement markers reduce a vehicle's lateral placement variance and driver stress at night in wet weather. (*5, 7*) Accident analysis studies showed that when painted centerlines were replaced with raised pavement markers, there was reduction of approximately 0.31 accidents per million vehicle-kilometres (0.50 accidents per million vehicle-miles). (*8*) In areas with no snow and where markers have a service life of 5 years and cost less than $2,500 per kilometre ($4,000 per mile) to install, raised pavement markers are cost beneficial at an ADT volume of 3,000 vehicles.

Because of the high initial cost of raised pavement markers, especially the snowplowable types, many highway agencies have supplemented standard painted centerlines with retroreflective raised pavement markers every 24.4 m (80 ft) to develop an all-weather delineation system at minimum cost. The cost of such a supplemental system, $620 to $930 per lane kilometre ($1,000 to $1,500 per lane mile), is considerably lower than the cost of complete replacement. In the human factors and traffic performance studies, hazards were reduced 30 to 40 percent with this type of treatment. (*5, 7*)

The traffic performance study indicated that raised pavement markers are more effective than postmounted delineators on isolated horizontal curves. (*7*) Raised pavement markers provide steering guidance in the area near the driver where actual steering decisions are made and provide far-distance information needed by drivers to anticipate road alinement changes. Raised markers also provide for a more accurate perception of the driving situation than do most other forms of supplemental delineation. Research suggests that one-way raised pavement markers along the outside of each driving path are more effective than two-way centerline markers for minimizing off-center driving on a two-way curved roadway. The cost effectiveness of such an installation depends on the particular site.

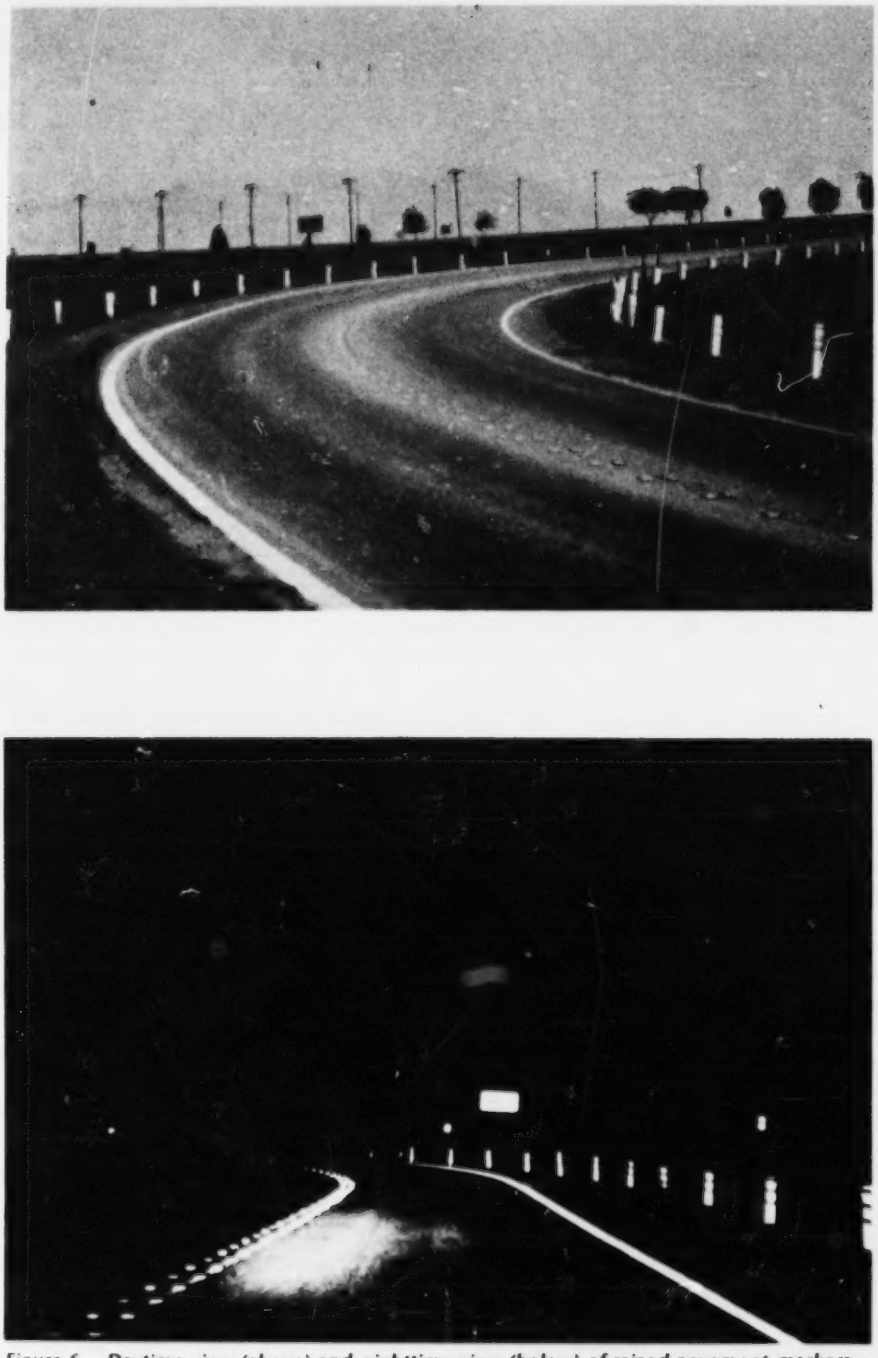

Figure 6.—Daytime view (above) and nighttime view (below) of raised pavement markers used for centerline delineation.

Postmounted delineators

Postmounted delineators of various shapes, colors, and reflective characteristics are used widely throughout the United States. These markings have proven especially effective at night and in adverse weather when standard paint markings are covered by snow or water. They provide the driver with a preview of roadway direction, but they are not very useful for near-roadway steering information because of their offset location.

Accident rates are significantly lower were postmounted delineators are used; a reduction of approximately 0.6 accidents per million vehicle-kilometres (1 accident per million vehicle-miles) has been demonstrated. (*8*) The postmounted delineator, therefore, can be very cost effective as an edge marking. If safety is the only benefit considered, such treatment is cost beneficial at all reasonable values of installation cost and service life for ADT volumes exceeding 1,000 vehicles. In many cases, depending on local service and cost specifics, such treatments can be justified for ADT volumes as low as 500 vehicles.

As with raised pavement markers, the selective use of postmounted delineators as a supplement to standard pavement striping appears to be cost effective for all weather conditions (fig. 7). In the traffic performance study, performance improved significantly with the use of postmounted delineators on horizontal curves. (*7*) The accident analysis study demonstrated a lower accident rate at isolated horizontal curves where postmounted delineators supplemented the standard paint markings than at matching-control sites without postmounted delineators. (*8*) However, the sample size was too small to make a definitive conclusion.

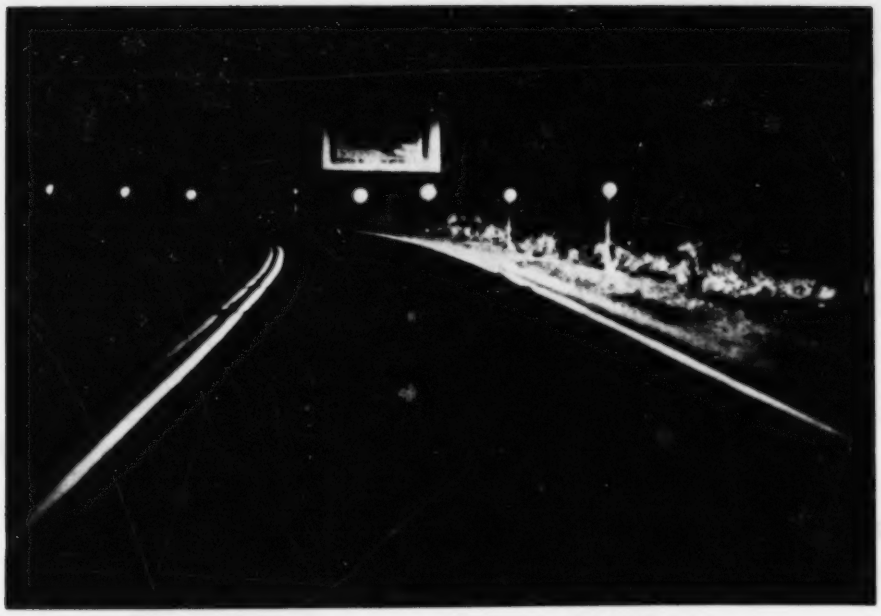

Figure 7.—Painted edgelines supplemented with postmounted delineators.

Discussion of Findings

Based on the findings of this research program, it was concluded that centerline delineation is highly cost beneficial and should be used on any paved roadway that carries two or more lanes of through traffic. If a low-volume roadway warrants the expenditure for paving and maintaining that surface, it certainly warrants the additional annual expenditure of $30 a kilometre ($50 a mile) to maintain a centerline and reduce accidents 30 percent. Delineation of the outside edge of the travel lane is also highly desirable, especially for roads wider than 6 m (20 ft). Postmounted delineators are somewhat more effective for this application than edgelines, but both treatments are especially useful for roadways with high ADT volumes.

There is substantial evidence that delineation provides important guidance information to motorists, especially when visibility decreases due to adverse weather or nighttime conditions, and that there is little difference in effectiveness between various delineation materials and treatments as long as there is adequate contrast and enough continuity to the delineation to inform the driver of roadway direction. For this reason, least life-cost systems are usually preferred for most roadway situations.

The above conclusions do not rule out the need for delineation in excess of minimum standards at certain locations. For example, standard delineation systems supplemented by other delineation treatments to improve traffic flow for nonstandard, high-hazard situations are generally very effective and result in great safety benefits. Among the critical locations that should be considered for such supplemental treatments are isolated horizontal curves, areas where driver expectation is different from what is actually encountered, and areas frequently subject to reduced visibility from dense fog, blowing snow, or heavy rain.

The state of the art is well enough established to provide minimum standard delineation treatments for general situations in terms of color, configuration, and contrast. General guidance is available to determine when these standard treatments should be supplemented. Better definition and solutions for determining where, when, and how much delineation should be provided are still needed. However, much of the remaining effort depends on local problemsolving for unique applications. This research program has developed the tools needed to evaluate alternative solutions.

In general, it can be concluded that wider application of conventional delineation treatments is desirable and will significantly reduce accidents.

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(12) Tom Miller, "Optimization of Traffic Lane Delineation," Report No. FHWA-TS-77-200, *Federal Highway Administration and California Department of Transportation*, Washington, D.C., December 1976.

[2] Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

Richard N. Schwab is an electrical engineer in the Environmental Division, Office of Research, Federal Highway Administration. He has been with FHWA since 1962 and is project manager for FCP Project 1L "Improving Traffic Operations During Adverse Environmental Conditions." His technical background is physiological optics and sensory capability of the human visual system. His work at FHWA has involved establishing the effects of adverse environmental conditions such as darkness, fog, snow, dust, and rain on traffic flow and highway safety and developing methods for overcoming these effects. Prior to joining FHWA, he was a graduate assistant on the staff of the Institute for Research in Vision at Ohio State University.

Donald G. Capelle is a vice president in charge of the Operations Division of Alan M. Voorhees & Associates (AMV). In this role he has been involved in a variety of traffic operations research related to roadway delineation, freeway traffic management, traffic safety, and traffic flow theory. Prior to joining AMV, Dr. Capelle was a traffic research engineer and a professor at Texas A&M University.

Development and Testing of Advanced Control Strategies in the Urban Traffic Control System

by
John MacGowan and Iris J. Fullerton

This is the second in a series of articles that traces the evolution and accomplishments of the Urban Traffic Control System (UTCS) research project. This series highlights the research activities undertaken in the past 10 years and stresses the potential application and anticipated benefits accruing from this major research project. The first article, published in *Public Roads,* vol. 43, No. 2, September 1979, described the development of offline signal timing programs, the provisions of the real-world fully instrumented laboratory established in Washington, D.C., and the first generation control strategy. This second article summarizes the hardware and strategies of the Bus Priority System (BPS), describes the second and third generation software systems, evaluates the various strategies, and presents research in traffic simulation and vehicle detection. The next article will present the technical accomplishments of the UTCS research project.

Bus Priority System

The Bus Priority System (BPS) study extended the original scope of the Urban Traffic Control System (UTCS) project. (*1*)[1] Operational hardware and computer software were developed to reduce bus delays in an urban network by providing preferential treatment to buses at signalized intersections. The test network consisted of 144 installations at 72 approaches of 34 intersections.

BPS hardware

The BPS configuration is shown on the flow diagram in figure 1. The system uses near-field radio frequency transmissions, a buried single-turn loop antenna, and a standard radio receiver circuit design.

The design configuration consists of an antenna buried in a single saw-cut across all lanes of an approach. A typical installation of bus antennas in relation to vehicle detector loops is shown in figure 2.

The bus detector transmitting equipment consists of the operator control lever and the radio transmitter located underneath the bus. The control lever hardware mounted on the steering wheel column is a modified turn indicator switch. The switch's position dictates which one of two frequencies (stop or through) is transmitted by the bus-mounted transmitter.

The bus detector receiver is a simple design consisting of two independent parallel channels containing an amplifier, a detector, and output circuits. (*2*) It is packaged into a plug-in module the same size and shape as the UTCS communication transmitters and receivers and operates from the same power supplies.

[1] Italic numbers in parentheses identify references on page 104.

The bus detector receiver output signals connect to a three-frequency-shift-keyed (FSK) communications transmitter that sends the appropriate signal over leased telephone lines to a bus detector communications receiver located at the computer center.

A BPS-dedicated computer in the control center was used for BPS operation and shared the UTCS computer memory. The BPS supplied the correct traffic signal interval timing and relied on the UTCS for signal advance pulses, as well as for all other control functions.

The field hardware operated reliably during a test period of 81 weeks; an onboard transmitter, switch, or power supply had a mean time between failure (MTBF) rate of approximately 47,700 hours.

BPS strategies

The original objective of the BPS algorithm was to increase passenger throughput and decrease passenger delay without disrupting other traffic. (*3*)

The method of granting priority to buses at intersections is shown in a simplified flow diagram (fig. 3). If there is a minimum net intersection gain[2] of 2 minutes by extending the green time and allowing the bus to pass through the intersection, then the green time would be extended for up to 10 seconds.

Because this algorithm was very conservative, the delay was reduced for relatively few buses. Also, assumptions had to be made to satisfy algorithm requirements for data that were not directly observable from the surveillance system. This reduced the effectiveness of the BPS. (*3*) It was also assumed that bus operators placed the "stop/through" switch in the correct position.

Recognizing these shortcomings, a second algorithm was developed for providing preferential treatment to buses at an intersection.

The new algorithm was a preemptive strategy using the same bus detector configuration. In addition to the green time extension, red time could be truncated. A bus arriving at the upstream detector during the red phase was immediately granted an early green signal, provided the cross street minimum green time had been achieved. A simplified flow chart of this priority method is shown in figure 4. No provision was made to correct extended or truncated offsets.

[2]Intersection gain is bus passenger gain (bus passenger benefits) plus car passenger gain (experienced by cars in the same phase with the bus) minus the cross street car passenger loss due to lost green time.

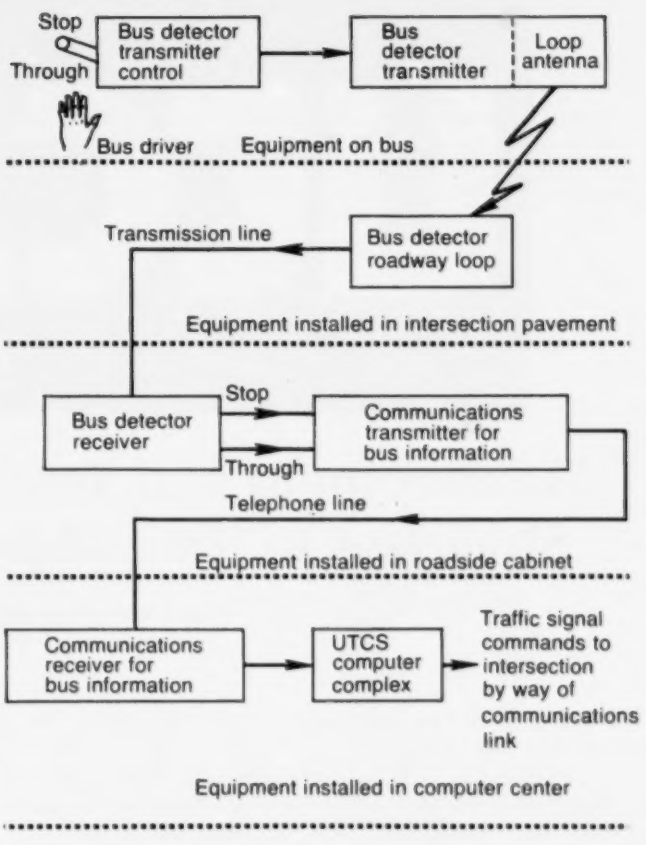
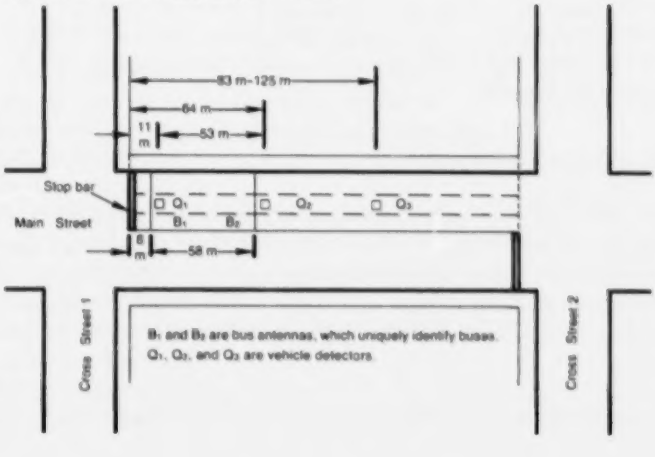


Figure 1.—BPS signal flow block diagram.

Figure 2.—BPS detector placement.



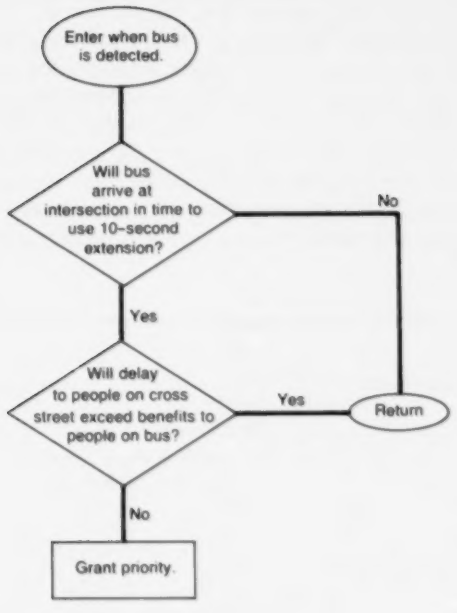
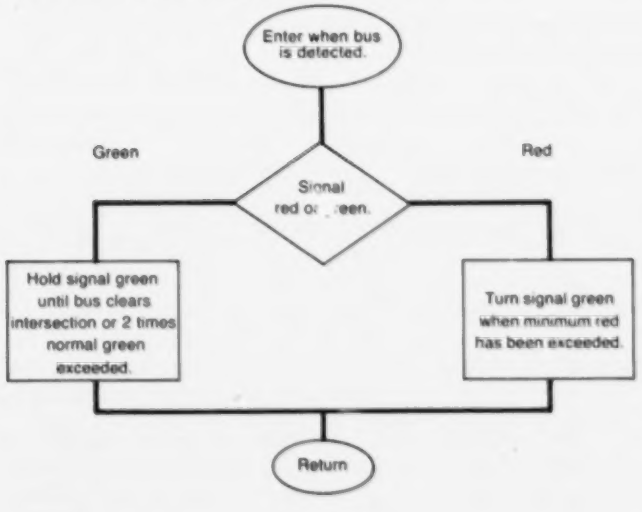

Enter when bus is detected.
Will bus arrive at intersection in time to use 10-second extension?
No
Yes
Will delay to people on cross street exceed benefits to people on bus?
Yes
Return
No
Grant priority.

Figure 3.—First BPS algorithm flow chart.

Figure 4.—Second BPS algorithm flow chart.


Enter when bus is detected.
Green
Red
Signal red or green.
Hold signal green until bus clears intersection or 2 times normal green exceeded.
Turn signal green when minimum red has been exceeded.
Return

Two significant modifications were made during field testing. The first allowed the offset to be corrected to the desired offset over a number of signal cycles when an intersection became inhibited from BPS operation.

The second modification introduced a vehicle saturation threshold to inhibit an intersection for bus operation. The saturation threshold was based on the measured flow (a function of vehicle volume and occupancy). If an approach to the intersection carried flow exceeding the threshold, then BPS would become inhibited at that intersection.

Simulation and evaluation

A series of simulation tests using the NETSIM model, which is discussed later in this article, preceded field testing and evaluation of the revised algorithm. During this phase, the new algorithm was incorporated into the model and experiments ascertained the impact of the new strategy on buses and other vehicles. The simulation tests showed that when buses preempted other traffic at signals there was approximately a 20 percent decrease in net bus delay; however, speed of other traffic was decreased by approximately 10 percent.

Numerical results from an onstreet evaluation, including the impact on the overall vehicle population, are presented in table 1. (4) The results show relatively small degradation to the total system. Tables 2 and 3 indicate that a preemptive strategy will decrease bus travel time. The magnitude of both positive and negative values in these tables indicates the results are inconsistent. During peak periods, buses traveling opposite peak direction were as likely to trigger a preemption as fully loaded buses traveling in the peak direction.

The northbound approach of the 18th Street and Pennsylvania Avenue intersection in Washington, D.C., shows significant bus delay during the morning peak (table 2). This was caused by continuous high pedestrian movement across Pennsylvania Avenue during 18th Street green time extensions, which blocked a major right turn bus movement. The relationship between pedestrians and the preferential treatment of buses must be considered in future applications.

Bus travel time was reduced nearly 3 percent on the shorter route (Route No. 1 in table 3) where the ratio of instrumented approaches to noninstrumented approaches was higher than on longer routes (Route No. 2 in table 3). Frequent noninstrumented approaches on Route No. 2 probably caused the overshadowing of positive systemwide results.

Bus travel time may be reduced at midday with minor impact on other vehicles, probably because of the less critical nature of signal offsets at that time.

Second Generation Control System

The second generation control (2-GC) strategy is a real-time, online system that computes and implements signal timing plans based on surveillance data and predicted changes. (5) The 2-GC program, designated TANSTP (Traffic Adaptive Network Signal Timing Program), retained many features of the first generation control (1-GC) UTCS software and hardware, such as the basic executive logic, surveillance system, controller timekeeping, man/machine interface, selection of prestored timing plans through the manual or time-of-day modes, and hardware failure detection. The optimization process is repeated at 5-minute intervals. Although new timing plans were available, they could not be implemented more often than every 10 minutes to avoid developing control parameters based on conditions that exist during signal timing transition.

Table 1.—BPS impact on total vehicle movements

Time	Vehicle minutes of travel			Vehicle minutes of delay			Derived average speed (veh-miles per veh-mins)		
	BPS	Base	Net	BPS	Base	Net	BPS	Base	Net
A.M.	13,087	12,774	−313[1]	8,324	8,115	−209	6.86	7.16	−.30
Midday	12,487	12,414	−73	8,215	8,160	−55	6.39	6.52	−.13
P.M.	12,232	12,238	+6	7,844	7,818	−26	6.68	6.78	−.10

[1] Negative values indicate a degradation in the system as compared to the base system; positive values indicate an improvement.

1 veh-mile = 1.6 veh-km

Table 2.—Intersections' performance as measured by bus travel time from the upstream detector through the intersection

Approach	Time	Intersection 18th & Pennsylvania (Mean time)			Intersection 14th & K (Mean time)			Intersection Wisconsin & Macomb (Mean time)		
		with XMTR[1]	without XMTR	Difference	with XMTR	without XMTR	Difference	with XMTR	without XMTR	Difference
		Minutes	*Minutes*	*Percent*	*Minutes*	*Minutes*	*Percent*	*Minutes*	*Minutes*	*Percent*
Northbound	A.M.	1.09	.92	−19.1[2]	.82	.95	+14.9	.34	.31	− 7.3
	Midday	1.09	1.34	+18.5	.96	.92	− 5.1	.44	.47	+ 4.9
	P.M.	1.35	1.44	+ 5.8	.98	1.13	+13.6	.37	.39	+ 6.2
Eastbound	A.M.	.84	.96	+12.8	.66	.72	+ 8.5	—	—	—
	Midday	.69	.76	+ 9.6	.66	.76	+12.5	—	—	—
	P.M.	.82	.82	0.0	.78	.80	+ 1.9	—	—	—
Southbound	A.M.	—	—	—	1.24	1.37	+ 9.2	.34	.41	+15.3
	Midday	—	—	—	1.05	1.30	+19.5	.53	.42	−27.6
	P.M.	—	—	—	1.25	1.37	+ 8.4	.35	.43	+18.8
Westbound	A.M.	.75	.70	− 6.5	.56	.63	+11.8	—	—	—
	Midday	.48	.83	+42.1	.53	.64	+17.6	—	—	—
	P.M.	.79	.73	− 8.1	.62	.74	+17.1	—	—	—
Average (all approaches)				+ 5.9			+10.2			+ 2.6

[1] XMTR = BPS transmitter.

[2] Negative values indicate degradation of BPS-equipped buses; positive values indicate improved travel time for BPS-equipped buses.

Table 3.—Route performance as measured by bus travel time along the route

Approach	Time	Route No. 1 Distance	1 (Mean time) with XMTR[1]	1 (Mean time) without XMTR	1 Difference	Route No. 2 Distance	2 (Mean time) with XMTR	2 (Mean time) without XMTR	2 Difference
		km	*Minutes*	*Minutes*	*Percent*	*km*	*Minutes*	*Minutes*	*Percent*
Eastbound	A.M.	1.5	7.92	8.46	+6.38[2]	5.6	24.11	23.56	−2.33
	Midday		8.88	8.59	−3.37		23.70	23.08	−2.66
	P.M.		10.00	10.24	+2.34		25.50	25.64	+0.54
Westbound	A.M.	1.3	6.45	6.79	+5.00	5.6	22.78	22.12	−2.98
	Midday		7.33	7.44	+1.47		23.93	23.99	+0.25
	P.M.		9.96	9.11	+1.64		27.34	26.50	−3.16
Average (all approaches)					+2.91				−1.73

[1] XMTR=BPS transmitter.
[2] Positive values indicate improved travel time for BPS-equipped buses; negative values indicate degradation of BPS-equipped buses.

1 km=0.62 mile

Operational features

The 2-GC software provides the following features:

- Optimization algorithm—The OPTIMIZ module of the SIGOP program was selected because of its short execution time, moderate computer memory storage requirement, and its simple and explicit mathematical optimization function (a quadratic equation of stops and delay terms).
- Traffic prediction model—Selected to predict the volume and speed as a function of historical volume and speed so that computed timing plans do not slow down current flow conditions.
- Subnetwork configuration models—The 2-GC software can improve optimization efficiency by dynamically reducing the total network into subnetworks on the basis of prevailing traffic conditions.
- Critical intersection control (CIC)—The CIC routine for 2-GC involves changing the cycle splits and offsets for certain intersections once per cycle as a function of the demand (computed values of queues and volume on these links).
- Transition model—The transition from a current timing plan to a newly computed timing plan is optimized by determining the value of a parameter, which when added to the new offset at each intersection will minimize the transition time.

Third Generation Control System

The third generation control (3-GC) UTCS software was developed to implement and evaluate a fully responsive, online traffic control system. (6) The 3-GC software was designed to permit the cycle length of each controller to vary from cycle to cycle. This design approach differentiates 3-GC from those control systems that provide a sequence of fixed-time, cycle-based signal timings. The 3-GC software system interfaces with existing field controller hardware; therefore, the local dial system remains intact and provides a backup (standby mode) for the computer-based system. Both 1-GC time-of-day and manual modes can be used.

Operational features

The unique operational features of 3-GC software provide for the following:

- Signal timing is computed for all controllers at least once every 3½ minutes.
- Signal offset and split are determined to minimize vehicle delay and stops along each approach and provide networkwide coordination.
- Congestion paths are identified and the control algorithm is specifically designed to service a congested condition while the condition persists.

• Transition routines are not needed because the system is constantly in a state of transition as it responds to changing traffic conditions.

Optimization algorithm

In the 3-GC traffic responsive mode, signal timing plans are developed by either of two optimization algorithms: The Cycle-Free Responsive Algorithm for Network Optimization (CYRANO) provides signal timing for undersaturated conditions, and the Congested Intersection Control coupled with Queue Management Control (CIC/QMC) provides areawide signal timing for saturated conditions on congestion paths.

Signal coordination is accomplished in CYRANO by implementing a coarse simulation of traffic flow and then systematically adjusting the signal settings at each controller to minimize disutility (a linear combination of vehicle stops and delay aggregated over all approaches). The objectives for the CIC/QMC are to maximize throughput at the congested critical intersections to provide acceptable service for the two congested competing demands and to manage the queue formation along the paths leading to the critical intersection so that the intersection is blocked and green time is fully utilized.

Evaluation of UTCS Software

The evaluation of UTCS included an intensive two-phase study to determine the effectiveness of the three generations of control systems and an evaluation of the 1-GC system recently installed in New Orleans, La.

Phase I evaluation

The phase I evaluation, conducted from March 1974 to November 1974, compared the 1-GC strategies and the existing Washington, D.C., three-dial, pretimed traffic control system. (*4*) All the traffic control plans were programed for the computer and were implemented with the computer as the master controller. This UTCS/BPS evaluation network included 114 signal-controlled intersections in central Washington, D.C., and along two primary arterials.

In the phase I evaluation, data were collected from the electronic surveillance system and from four survey vehicles traveling through the control area. To evaluate the BPS control alternatives, special studies were conducted to measure bus activity.

The results of the phase I evaluation will be fully discussed in the next article in this series. The major conclusions are summarized below.

• Generally the traffic responsive (TRSP) control alternative compared favorably with all other control alternatives.

• The CIC operations distributed delay equally on the approaches to a given intersection, but had little effect on overall system operations.

• The BPS algorithm worked well when the offset from the previous intersections was not critical. Overall traffic delays were increased by only 0.3 to 2.5 percent, so the effect on motorists was minimal. Bus delays were reduced as much as 42 percent at one intersection; however, most delay savings were lost because not all intersections along the bus route were equipped to operate with BPS control.

• The measures of effectiveness calculated from the surveillance system compared well with those obtained from the moving car surveys.

Phase II evaluation

Phase II of the evaluation study, conducted from February 1976 to May 1976, evaluated 2-GC and 3-GC (CYRANO only) strategies. (*7*) The TRSP and the Washington, D.C., three-dial control alternatives of phase I were reevaluated.

The evaluation methodology used for phase I was changed for phase II because the higher order measures of effectiveness, including delay, would not be generated when the 2-GC and 3-GC were online. The phase II evaluation was based on travel time measurements made by moving survey vehicles and by relying on the surveillance system for volume measurements only.

Phase II evaluation results will be fully discussed in the next article of this series. The major conclusions are summarized below.

• The 2-GC strategy was comparable to the Washington, D.C., three-dial alternative, except for the evening peak period in one area of the network where there was a relative degradation in performance.

• The 3-GC strategy was the least effective of the new concepts tested, indicating that more research is needed before it can be thoroughly tested.

• Although the TRSP alternative did not compare as favorably to the Washington, D.C., three-dial alternative

as it did in phase I, the changes in the traffic patterns because of the completion of the subway construction may account for the TRSP decline in effectiveness.

New Orleans evaluation

The city of New Orleans, La., installed a new computer-based signal system that uses the concepts developed for the UTCS in Washington, D.C. The New Orleans system is one of the first to use the FORTRAN IV version of 1-GC UTCS.

Basically, the evaluation focused on onstreet measures of performance and system operation measures of performance. (*8*)

Benefits of a computer-based system include the following:

- A map display identifies abnormal traffic conditions and a control console revises signal settings in real time.
- Summary performance reports identify long term changes in traffic demand and indicate when to generate new signal plans or modify existing ones.
- New street timing plans can be debugged (tested onstreet) efficiently by defining a new plan in the data base.
- Hardware monitoring immediately identifies detector or controller failure, permitting rapid response to equipment malfunctions. Hardware monitoring will also reduce false maintenance calls, allowing more efficient scheduling of maintenance crews.

General conclusions drawn from the evaluation include the following:

- There were improvements over the existing plans of 8.8 percent for time-of-day and 8.5 percent for TRSP.
- The initial cost of a computer-based system in New Orleans was approximately 80 percent greater than the cost of a conventional system.
- Only minor differences in operations and maintenance costs were found, except that a computer-based system requires a full-time operator and a computer maintenance contract.

Research in Traffic Simulation

Shortly after the UTCS project was initiated, it became evident that an analytical model was needed to test and evaluate alternative network strategies in a "laboratory" prior to committing time and resources to installing the new control strategies in the Washington, D.C., network. The Federal Highway Administration (FHWA) sponsored the development of such a model. (9) It was originally designated UTCS-1 because of its relation to the UTCS project but was renamed NETSIM.

Model development and validation

The NETSIM model describes a street network in a series of interconnected links and nodes. Each link carries traffic in one direction and the nodes are link intersections. Traffic is processed in a series of short time-steps subject to varying forms of traffic control. Vehicle movements are very detailed in NETSIM, and the model can treat most of the major forms of traffic control found in U.S. cities. It is written in FORTRAN IV language for operation in virtually any large computer.

The model was first validated by tests in 1971 and 1973 when key outputs were compared with equivalent field data collected by aerial photography. In 1976, capabilities to estimate vehicle fuel consumption and vehicle emissions were added to the model. New logic for traffic-actuated signals was introduced into the model in 1977.

NETSIM applications

Some of the applications of NETSIM during the UTCS project are listed below.

- Tested the transition techniques between different signal settings.
- Compared traffic signal optimization models.
- Analyzed CIC schemes.
- Analyzed potentials of alternative methods of giving preferential treatment to buses at signalized urban intersections.

Research in Vehicle Detection

Several research studies in the UTCS project addressed detector technology. Installation techniques were increased, detector location for computer-based control systems was determined, and innovative detectors, such as the magnetic vehicle detector and self-powered vehicle detector, were developed.

Loop detector tests

The inductive loop detector used in the UTCS project consisted of a few turns of wire embedded in the roadway pavement. The wires were connected to an electronic component located in a cabinet at the side of the road. The detector was designed so that a vehicle passing over the loop in the pavement modified the magnetic flux around the resonantly tuned loops of wire. This increased or decreased the inductance so that a change in resonant frequency, impedance, amplitude, or phase was detected by the transistorized roadside unit and transmitted to an amplifying or relay circuit.

An extensive loop detector testing program was part of the UTCS facility specification effort to determine the accuracy, reliability, and maintainability of fixed-frequency inductive loop detectors. (*10*)

Installation techniques

Because vehicle detectors are key elements in surveillance subsystems, their performance and system cost can have a major impact on traffic control systems. FHWA developed an improved vehicle detector with low installation cost. (*11*)

Detector location study

A special study was conducted to determine the most effective location and placement of detectors for the advanced UTCS traffic control strategies. (*12*) The following conclusions were made:

- 1-GC and 2-GC CIC algorithms require detectors on all major approaches to a CIC-controlled intersection.
- 1-GC requires detectors on approximately every fourth link of an arterial and every third link of a network grid.
- 2-GC requires detectors on every other link on streets with uniform flows and more frequently on streets with fluctuating flows.
- A detector located in one "critical" lane can usually identify demand on a multilane approach.
- Detectors should be placed approximately midblock or 61 m (200 ft) upstream of the stopline.

Magnetic gradient vehicle detector (MGVD)

The MGVD consists of a transducer measuring 2 083 by 38.1 by 12.7 mm (82 by 1.5 by 0.5 in), cables, and roadside detector electronics. (*13*) The transducer is installed in a slot in the pavement, at right angles to the direction of vehicle travel.

In a 1974 test conducted by FHWA personnel, the performance of the MGVD was compared with inductive loop detectors and no significant difference was found. FHWA later developed a phase mode vehicle detector (PMVD), instead of the amplitude mode, but still using the MGVD transducer. The PMVD requires less than one-tenth of the power and is less costly to build than the amplitude mode. In 1976, four PMVD's were installed in the UTCS network using the MGVD transducers. An evaluation of the performance of this set of electronics resulted in a design change to the transducer.

The improved models of the MGVD/PMVD are being evaluated under the Federally Coordinated Program of Highway Research and Development, Project 2L "Detection and Communication for Traffic Systems."

Self-powered vehicle detector (SPVD)

An SPVD was developed to significantly reduce the time, cost, and traffic disruption associated with detector installation by eliminating the need to cut slots in the pavement for connecting cables between the detector and the roadside. (*14*)

The SPVD roadway unit consists of an internal power source, transducer, and radio transmitter with loop antenna. It is packaged in a cylinder and is readily removable for servicing. The power source is a lantern battery with a minimum life expectancy of 1 year. The transmitter with loop antenna has an effective range of 152 m (500 ft) from the detector to the roadside equipment.

An evaluation of the SPVD prototypes revealed major design problems with the magnetometers (a portion of the transducer) and telemetry link. Improved production prototype models of the SPVD were developed and are currently being evaluated in the field under Project 2L.

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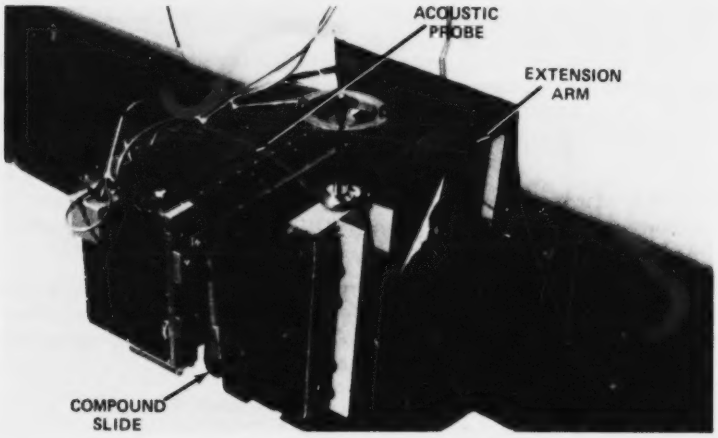
John MacGowan is a highway research engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been with FHWA since 1970, first as a staff member on the UTCS research team and then as the project manager of FCP Project 2B "Development and Testing of Advanced Control Strategies in the Urban Traffic Control System." Mr. MacGowan is currently the project manager of FCP Project 2M "Arterial Flow and Control." This research concerns arterial capacity, signal control for arterials, and bus signal priority systems.

Iris J. Fullerton is a Senior Public Systems Specialist for JHK & Associates. She specializes in research synthesis, systems evaluation, engineering training, and the communication and implementation of applied research. She is currently principal investigator for the FHWA project "Roadway Delineation Practices" and is participating in the National Cooperative Highway Research Program (NCHRP) Project 3-28 to develop an improved Highway Capacity Manual. Ms. Fullerton participated in the evaluation of UTCS control strategies and NCHRP Project 3-18(3) "Cost Effectiveness Methodology for Evaluation of Signalized Street Network Surveillance and Control Systems." She coauthored *Urban Street Design*, the student textbook for the FHWA's Urban Transportation Operations Training Program.

The Evaluation of a Noncontact Profiling System Using the Acoustic Probe



by
James C. Wambold[1]

This article evaluates the Rapid Travel Profilometer and describes how the four States owning a profilometer make use of it. From the States' experiences, filter and gain requirements for a profiling system are established. The operating principle of the noncontact profiling system is described, as are the following components: Acoustic probe, acoustic probe signal conditioning, vehicle body motion accelerometer, and the signal processing. A system evaluation procedure is presented with a discussion of operation experience and the difficulties encountered. The present system is usable; however, because of the many bench-type components in the system, further development to adapt it to over-the-road environment is recommended.

[1] This article is in part taken from "The Evaluation of a Noncontact Profiling System Using the Acoustic Probe," by James C. Wambold, Report No. FHWA-RD-78-43, *Federal Highway Administration*, Washington, D.C., March 1978.

Introduction

This article discusses a noncontact acoustic probe (AP), its phase measuring equipment, an accelerometer, and commercial integrating and summing analog equipment that were mounted and installed in a sports van to determine road profile. (*1*)[2]

The Rapid Travel Profilometer (RTP) uses two spring-loaded, road-following wheels instrumented with a linear potentiometer to measure relative displacement between the vehicle frame and the road surface of the two wheel tracks. (*2–4*) The RTP system is manufactured with instrumentation to measure vehicle frame motion. The frame motion is then added to the relative motions to yield two voltage signals which, in theory, are the road profiles of the wheel paths.

[2] Italic numbers in parentheses identify references on page 113.

The RTP system uses two accelerometers, each mounted on the frame over one of the follower wheels, to measure the vehicle frame motion. These acceleration signals are double integrated by an onboard analog computer and then added to the relative displacement signal to obtain the road profile signal (fig. 1). To accommodate various wavelengths at various road amplitudes, the RTP onboard computer is equipped with four selectable gains (0.2, 0.5, 1.0, and 2.0) and four selectable high pass filters (0.3, 0.6, 1.0, and 3.0 rad/sec).

In practice the amplitude is attenuated at the cutoff frequency, and there is phase shift of the signal as well. The amount of attenuation and phase shift depends on the order of the filter. Figure 2 is a plot of the typical phase shift and frequency response (amplitude magnification) of the four filters.

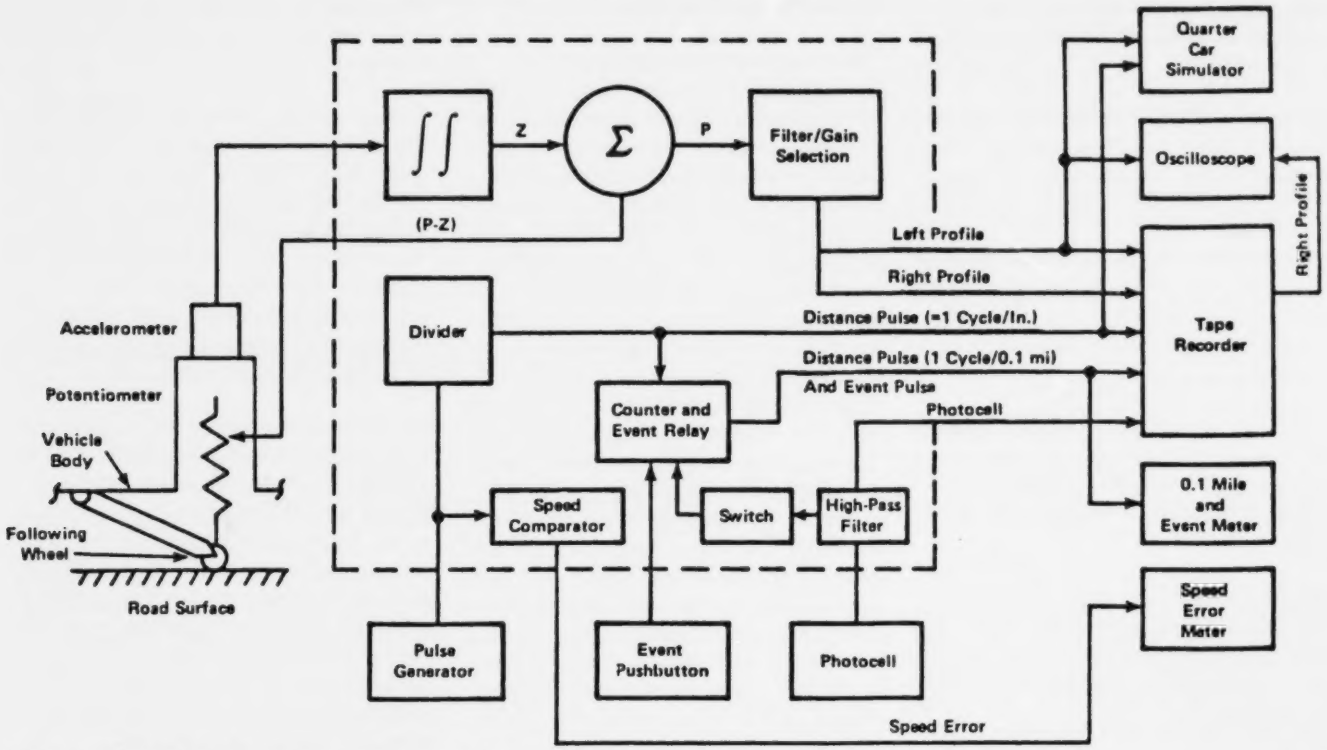


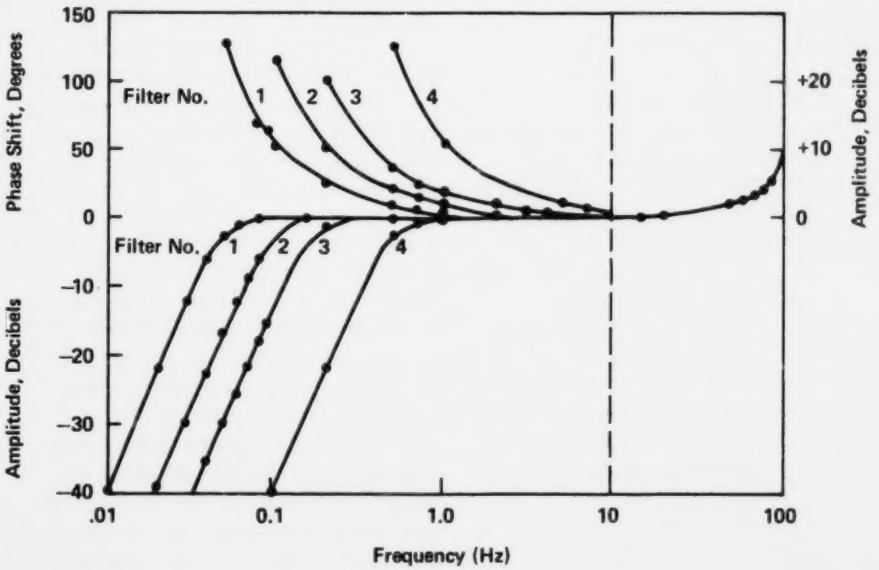
Figure 1.—Block diagram of measurement system for the RTP.

The main limitation of the RTP system is the road-following wheel. (*5, 6*) Because it is a mechanical dynamic system, frequency response and profiling speed are limited. Because of this weakness in the RTP, a noncontact probe was developed to replace the road-following wheel so the limitation on frequency response would be removed. (*1, 7*)

States Now Using the RTP

At present, Kentucky, Michigan, Pennsylvania, Texas, and West Virginia have RTP's. West Virginia has the newest RTP (purchased in the last year), in which all analog signals from the transducers are converted into digital signals and then all digital processing is performed onboard. All

Figure 2.—RTP system frequency response and phase shift.



the other States have all analog models and have had them for 10 years or more.

Kentucky and Pennsylvania also have quarter-car-simulators (QCS) onboard that simulate the Bureau of Public Roads (BPR) roughometer and use the profile signal as input. (*8*) All of the onboard processing is analog, including the QCS. Texas has analog processing equipment to obtain profiles, but no QCS. Michigan has replaced its analog processor and is conducting its processing digitally—some onboard and some in the laboratory. The report referenced in footnote 1 briefly reviews how each of these four States uses its RTP.

Based on what the four States are presently doing with their RTP's, any new profilometers developed should have a selection of filters and gains as follows:

Gain: 0.5, 1.0, and 2.0

High Pass Filter Cutoff Frequency: 0.3, 0.6, 1.0, and 3.14 rad/sec

These selections are the same as those of the RTP except the 0.2 gain and No. 4 filter are not used. Michigan omitted the 0.2 gain and replaced the 3.0 rad/sec with a new No. 4 filter of 3.14 rad/sec.

These settings are required for both analog and digital processing. For analog processing, they are hardware items; for digital processing, they could be hardware, software, or, more likely, a combination.

Noncontact Profiling System

The noncontact profiling system consists of four major parts: (1) The AP to measure relative displacement, (2) phase measuring equipment to produce a voltage proportional to the relative displacement, (3) the accelerometer to measure the acceleration of the probe support, and (4) the analog processing equipment to double integrate the acceleration signal to produce a voltage proportional to the probe support motion and then to sum this with the relative displacement signal to produce a voltage proportional to the profile. In addition to analog processing, digital processing is available. For digital processing, the relative displacement signal and the acceleration signal are converted from analog to digital in the laboratory, and then a digital signal of the profile is produced by digital integration and summing.

The AP consists of a sound generator and receiver that face the road surface so that the sound transmitted by the generator reaches the receiver after being reflected from the road surface. The time delay of the sound wave arriving at the receiver is a function of the probe height above the surface. Changes in time delay due to changes in road-to-probe distance can be measured with great precision if the resulting phase angle change is measured. A complete description of the AP is given in references 1 and 7.

A low g range (± 15 g's) accelerometer with a d.c. to 500 Hz frequency was mounted on the AP body to measure the probe body motion. A signal conditioning unit was also used to obtain accurate acceleration data from the accelerometer. A 1.0 rad/sec RC high pass filter was inserted into the system after the signal conditioning unit. This filter was used to provide an a.c. coupling so that d.c. offsets in the signals are removed (in particular, a d.c. signal resulting from the accelerometer not being vertical).

To use the acceleration and AP data, the acceleration must be integrated twice to give body motion, the body motion signal and AP signal must be scaled to the same value, and then these two signals can be summed to produce a profile signal. This processing was done with analog equipment onboard the profilometer and digitally in the laboratory.

The onboard processing equipment is an analog system comprised of various modular elements: (1) A mainframe that provides all necessary power supplies, (2) two analog signal conditioning modules that accept the acceleration and AP signals, (3) two integrating modules that double integrate the acceleration to provide body displacement, and (4) a summing module to sum the double integrations with the AP signal to produce the road profile signal.

The analog equipment, accelerometer system, AP and its measuring equipment, a fifth wheel for speed measurement, and a separate slider (a linear potentiometer to measure relative motion for calibration purposes) were calibrated and installed in a van. Power supplies, an analog tape recorder, and a strip chart recorder were also added. Figure 3 shows the complete system's wiring. The integrator modules were modified by placing a 1.5 meg resistor across the integrating capacitor to give a 2 Hz high pass filter. This kept the amplifiers from saturating.

System Evaluation Procedures

The following four-task testing procedure evaluated the performance of the AP and established the operational characteristics.

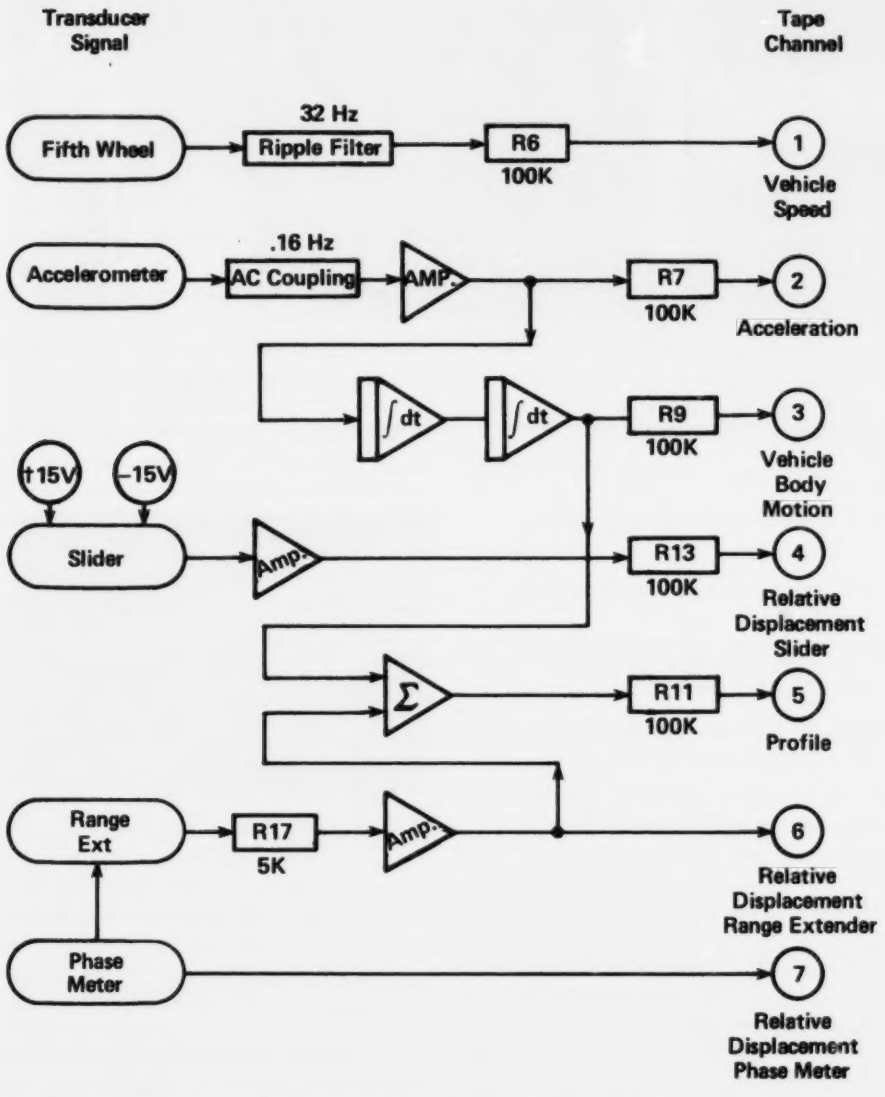


Figure 3.—Wiring of AP profiling system. R6, R7, R9, R11, R13, and R17 are trim potentiometers.

Task 1—Set up the AP for bench tests to evaluate precision and repeatability.

The laboratory evaluation included the following tests:

- Determine the phase shift and gains of the acoustic amplifiers and filters.
- Determine the sound levels.
- Test the linearity of the phase detector and range extender as a function of AP distance from the surface.
- Measure the footprint of the AP.

Task 2—Mount the AP, an accelerometer, and recording instrumentation on a vehicle to run exploratory tests.

The AP was mounted to the frame on the left side of a 1974 van. The AP was attached to the frame with a Secto travel compound slide so the probe could be positioned vertically and laterally. The AP was positioned to track in the path of the van's left wheels. Figure 4 shows the location of the AP mounting. Figure 5 shows the instrumentation mounted in the van.

A 0.5 by 15.2 m (1.5 by 50 ft) sandbox was built along the road at the Federal Highway Administration (FHWA) Fairbank Highway Research Station (FHRS), McLean, Va. Two plywood forms were made so sinusoidal wave shapes with wavelengths of 230 mm (9 in) at amplitudes of 10 and 20 mm (0.4 and 0.8 in) would be produced (fig. 6). The van was then driven with the AP extended over the sandbox.

These tests showed that, even at slow speeds, the van had enough body motion to invalidate the measurements. To account for body motion, an accelerometer was mounted on the AP body.

Figure 4.—1974 van with AP.

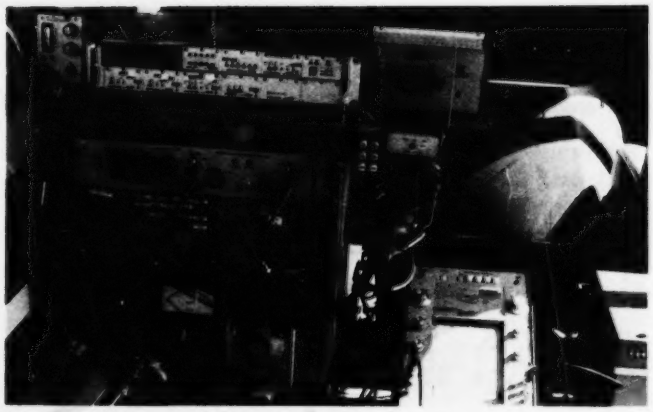


Figure 5.—Instrumentation mounted in the van.

Task 3—Run a dynamic calibration of the system.

To calibrate the van-accelerometer-probe system (AAP), a trailer hitch was added to the van and was connected by a rod to a hydraulic servo shaker. Tests were conducted at every 0.1 Hz from 0.5 to 1.0 Hz and at every 0.5 Hz from 1.0 to 7.0 Hz.

Task 4—Conduct road tests to determine repeatability and effects of testing variables.

For these tests, the system was operated on the road. First roads on the grounds of FHRS were profiled and then roads in the Washington, D.C., area were profiled, including the George Washington Memorial Parkway and the Dulles Access Road.

Preliminary Operating Experience and Difficulties

A number of tests were performed including static and dynamic laboratory tests and three sets of field tests to compare measured results with the RTP systems. (*1*) A summary of the difficulties encountered follows:

- Amplitude modulation of the sound signal.
- Irregular reflections of sound signals.
- Frequency response and slewing rate of phase meter.
- Signal loss and range extender dropout at some conditions.
- Components not able to withstand over-the-road use, in particular, vibration and wet weather.

However, the probe and the RTP road data compared favorably and the acoustic probe is capable of recording surface profiles at speeds of 26.8 m/sec (88 ft/sec) with a vertical height resolution of 0.25 mm (0.01 in).

The experiences and difficulties encountered by FHWA during the initial staff studies are given below.

Bench tests

The AP was tested and a report was issued describing the findings of the laboratory tests. A summary of the important findings is given here.

- The acoustic equipment had phase shifts that tracked within 2°, resulting in a distance error of about 0.25 mm (0.01 in).
- The system would experience dropouts with loss of signals, resulting in the range extender losing track of its count.
- The acoustic probe was blind to an object of one-half the wavelength of the acoustic signal.
- Sufficient information could not be found on the effects of and method to compensate for temperature variation.
- The system was quickly rendered inoperative by rain or wet pavements.

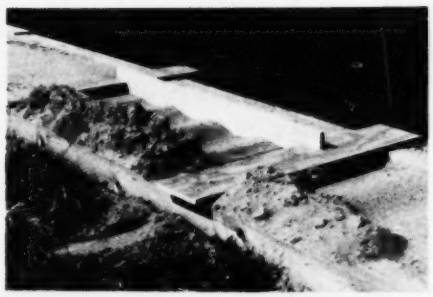


Figure 6.—View of the sandbox used to produce profiles.

In addition, it was concluded that "the noncontact probe as tested in the laboratory appears to present an interesting and promising technique for the measurement of distance between itself and a road." It is also recommended that a minimum field test program be undertaken with frequent data review by a committee and, following evaluation, a Request for Proposal be issued for the further development of the probe.[3]

Exploratory sandbox tests

The sand tests were intended to be exploratory and showed that the vehicle body motion must be known for any measurement. No qualitative results were obtained because body motions were not measured. The use of the sandbox is an excellent test method and is recommended for use in further tests, but the sandbox should be at least 30 m (100 ft) long.

Dynamic calibration

This test provided the most useful information. Unfortunately, the measurements made with a ruler were not made at every data point because these measurements were only made to calibrate the linear potentiometer (slider) which was used as the reference signal. Because the slider was only attached to the vehicle and not to the ground, ground contact was lost at higher frequencies, which invalidated the slide as a reference at higher frequencies.

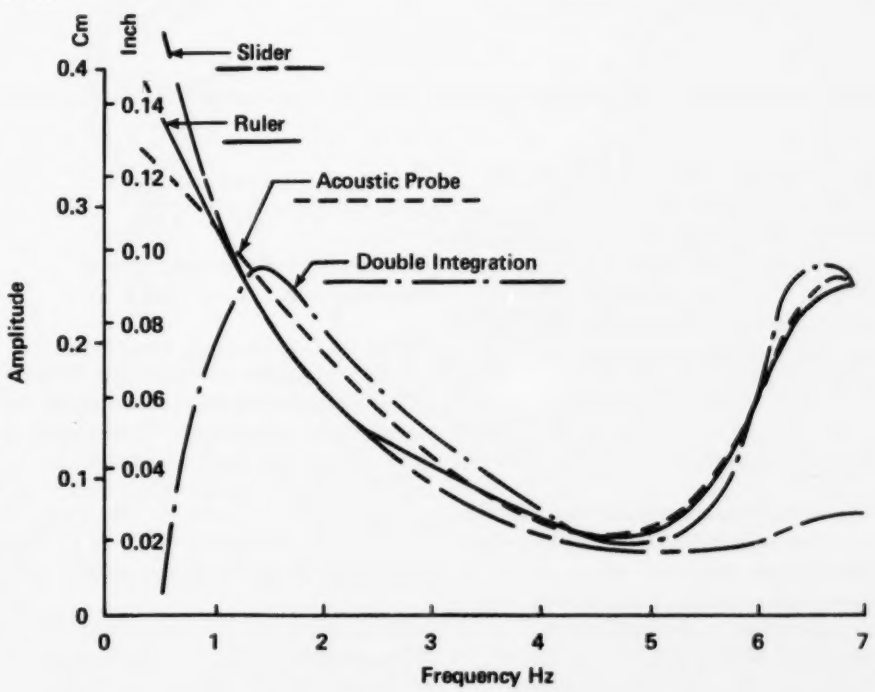
Amplitude levels

Both amplitude and phase shift were determined in the dynamic tests. Because of the many voltage dividers added to the system, the overall gains were not known and are determined from these calibration tests. Figure 7 is a result of matching the various signals to the ruler measurements. Good agreement of the slider signal is obtained up to about 5 Hz. Above this frequency, the effect of the slider losing ground contact is very obvious. The probe signal agrees very well with the ruler; in fact, at all data points, the probe signal passes through every point except 0.4 Hz. Similarly, the double integration signal agrees well above 1.5 Hz. Because the integrators were modified to create a high pass filter, the attenuation of the signal below 2 Hz is to be expected. In fact, the high pass filter modification was set for a 2 Hz cutoff.

Phase shift

Unlike the calibration of amplitude levels, the actual phase shift could not be determined because there is no reference signal for comparison. Figure 8 is a plot in which the slider is assumed to have no phase shift up to 2 Hz and the range extender and double integration signals are assumed to have no phase shift above 5 Hz. A smooth transition of the slider is assumed between 2 and 5 Hz. Theoretically, the acceleration signal is passed through a high pass RC filter set at 1.0 rad/sec (0.16 Hz) so that one would expect it to start at a 90° phase shift and go to 180° around 4 rad/sec (0.63 Hz). The shift between the range extender and integration does go to zero above 5 Hz. Thus, the sum of the two signals would be expected to add properly in this

Figure 7.—Comparison of the four displacement measurements over frequencies of 0.5 to 7 Hz.



[3] "Report on the Laboratory Evaluation of the Acoustic (Noncontact) Probe," by Joseph C. Leifer, Nov. 10, 1975. Unpublished report.

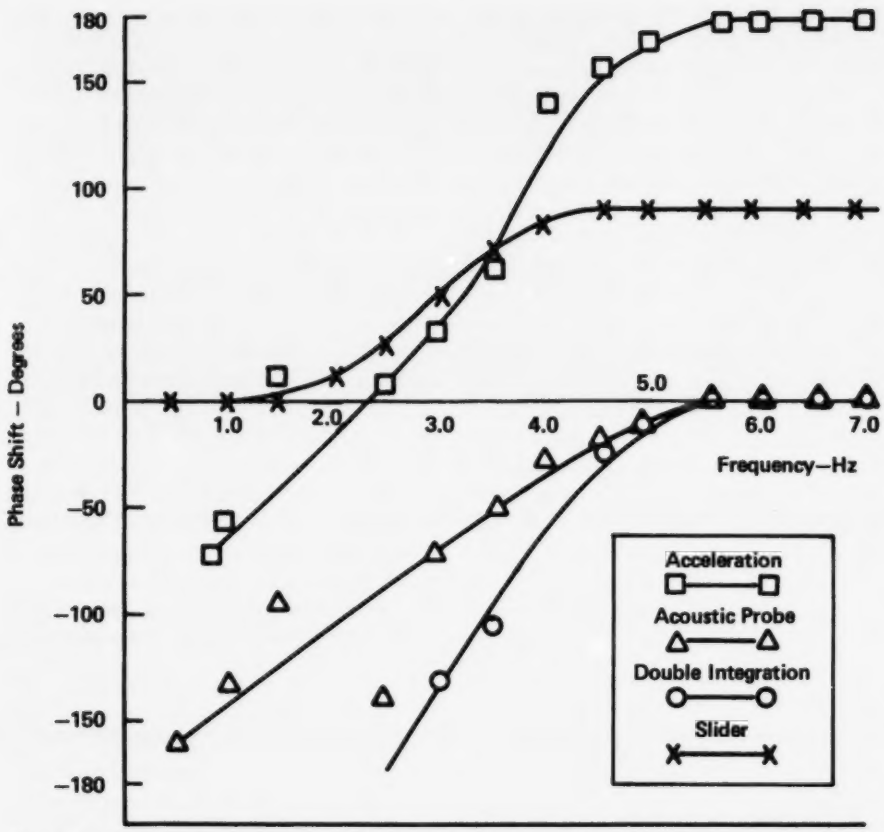


Figure 8.—Phase shift comparison with slider used as zero shift at low frequencies and the AP used as zero shift at high frequencies.

range. Below 5 Hz, the phase shift causes improper addition, but, again, this will be corrected by lowering the cutoff frequency from 2 Hz to 0.16 Hz or lower. In fact, there are phase shifts among all the signals measured and their sources must be determined.

Conclusions

The prototype of the AP was used in a series of in-house tests to (1) familiarize research and development (R&D) personnel with its operation and establish its operational characteristics, (2) evaluate it by static and dynamic testing, and (3) evaluate data processing methods.

Familiarization and operational experience

FHWA R&D personnel have now had 2 years of experience with the AP and about 1 year with the analog data processing equipment. Adequate experience has been obtained to evaluate the AP and analog data processing.

Evaluation of the acoustic probe

In evaluating the AP, one must remember that this is a prototype and many of the components are laboratory-type equipment and are not designed for on-the-road use. Although the prototype does not operate under certain environmental conditions, there was enough operating experience to evaluate the operating principle.

The physics of using the AP method to measure distance is sound, and testing has shown that the AP is able to measure distance accurately. Further, the AP can be made to work as part of a noncontact profilometer. There are some problems with the present prototype, but they can be solved. The present shortcomings of the probe follow, along with a discussion of possible solutions:

- Periodic signal loss results in the range extender losing track of its count.
- The footprint area is not well defined.
- Compensation for temperature is not made completely.
- Components are not made to withstand on-the-road use, particularly on wet roads.
- The AP cannot measure objects one-quarter the wavelength of the signal.
- There is signal interference from outside noise, such as aircraft, wind, and other traffic.

Although most of the dropout in earlier tests (laboratory and sandbox tests) was found to be a broken lead in a microphone cable, periodic dropout does occur; one reason is the AP's inability to measure objects one-quarter the wavelength of the signal. The causes of signal loss need to be studied further to minimize or eliminate signal loss either directly or in the computer processing. One method that seems to eliminate

signal loss is frequency modulation of the acoustic signal. This allows for a signal of much lower frequency (and longer wavelength) to be used with a much higher carrier frequency. The advantages of this include the following:

- The range extender could be eliminated because the signal wavelength could be adjusted to the required height. Then if the signal was lost, position would be known when the signal was reestablished.
- The carrier frequency could be set to optimize reflection and minimize signal loss.
- The higher frequency used to optimize reflection, along with the FM rather than AM signal, would greatly reduce the influence of noise.

Evaluation of data processing

The analog equipment cannot cope with any signal bias because double integration of even a small offset soon overloads the output. The initial filtering placed on the integrators solved this problem, but it also filtered out the signal in the frequency of most concern. The cutoff frequency will need to be lowered by an order of magnitude. Analog double integration is presently performed by the RTP and by the Simple Survey Profilometer (SSP). (9) In fact, the SSP uses the same analog equipment, so a satisfactory double integration is possible.

Acknowledgment

The author would like to thank the Electronic Instrumentation Group, Engineering Services Division, FHWA, for its assistance in the laboratory evaluation of the acoustic probe.

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(2) G. G. Balmer, "Road Roughness Technology: State of the Art," Report No. FHWA-RD-73-54, *Federal Highway Administration*, Washington, D.C., December 1973.

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(9) W. H. Park, J. C. Wambold, and A. D. Brickman, "Implementation of a Rating Procedure for Pavement Surface Roughness, Interim Report," Report PTI 7809, *The Pennsylvania Transportation Institute*, University Park, Pa., May 1974.

James C. Wambold is an associate professor of mechanical engineering at the Pennsylvania State University. He has been involved in research on vehicle/road interaction for the past 12 years. Dr. Wambold's interests include random vibration, vehicle dynamics, and ride comfort. He has also been involved in road roughness measurement. He recently spent a 9-month sabbatical leave with the Protective Systems Group, Structures and Applied Mechanics Division, Federal Highway Administration.

Laboratory Studies of Lane Occupancy Signs

by
Donald A. Gordon[1]

Introduction

Many signing problems may be studied conveniently in the laboratory. The advantage of laboratory research is that a large number of signs can be quickly and inexpensively investigated. Also, control can be maintained over illumination, sign pattern, and driver training and experience that is not possible in the operational situation. Drivers' opinions of the signs can be conveniently solicited and subtle differences in sign effectiveness can be revealed. For these reasons, laboratory tests are particularly useful for preliminary sign testing. However, the design and improvement of signs is best achieved by a balanced program including both laboratory and field research.[2] Whenever possible, the laboratory findings should be validated by field tests.

The Traffic Systems Division, Federal Highway Administration (FHWA), is now conducting an in-house study on the improvement of highway signs. Included are laboratory studies on the improvement of lane occupancy signs. Research on parking signs will be described in the next issue of *Public Roads*. In the lane occupancy sign studies, the experimental subjects viewed slides of conventional and novel signs and judged whether certain lane occupancy maneuvers were permitted. The effectiveness of each sign was determined by the speed and accuracy of the subjects' responses.

Lane Occupancy Signs

Lane occupancy (lane control) signs show the driver the particular lanes of a highway within which he or she may drive. Currently approved devices for lane control include signals showing a green downward-pointing arrow over permitted lanes and a red X over prohibited lanes. Fixed schedule signs that list occupancy

[1] This article is a condensation of the report "Regulatory Signs for Lane Occupancy and Parking," by D. A. Gordon, Report No. FHWA-RD-78-89, *Federal Highway Administration*, Washington, D.C., June 1978.

[2] T. Mast, "Alternative Techniques for Testing Highway Information Systems." Paper presented at the American Institute of Aeronautics and Astronautics Systems Conference, Arlington, Tex., Nov. 6-8, 1974.

information for the entire day are also used. In the lane occupancy sign studies, the following sign series were assessed: Series 1—fixed schedule signs now on the road, Series 2—printed message signs (without schedule), Series 3—signs showing the red X and green arrow symbols similar to the red and green "signals" approved by the Manual on Uniform Traffic Control Devices (MUTCD),[3] and Series 4—a design developed under FHWA sponsorship at Texas A&M University.[4] Forty subjects viewed these signs in the laboratory and judged whether the signs permitted certain lane maneuvers.

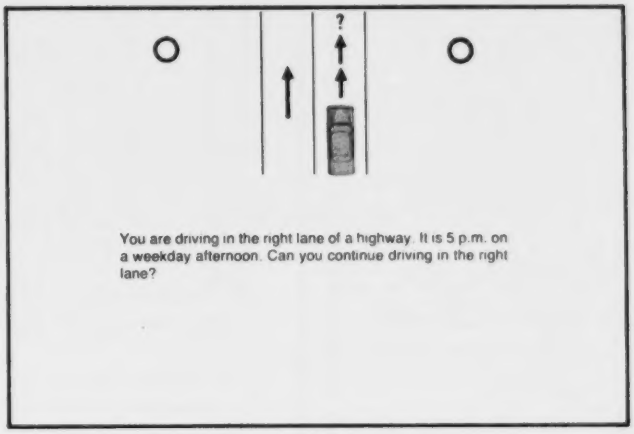


Figure 1.—A lane occupancy problem.

Subjects

Subjects for the study were recruited from the Virginia State Employment Office and from FHWA's Fairbank Highway Research Station (FHRS), McLean, Va. The sample of 40 subjects included an approximately equal number of males and females and a range of ages, education, and driving experience. All subjects had valid driving permits.

Equipment

Question booklets

Lane occupancy problems were presented to the subjects on library-style cards, fastened to rings (fig. 1). The problem stated the lane presently occupied by the driver and the time of day and asked whether a certain maneuver was permitted. Each problem was illustrated by a diagram of the road situation. When the subject said, "Ready," the road sign was projected on the screen. After answering a question, the subject turned to the next card. The same 22 questions were asked on each of the four sign series.

The sign series

Examples of the four sign series under consideration are shown in figure 2. Series 1 consisted of nine replicas of fixed schedule signs located around Chain Bridge, Va.; Telegraph Road, Va.; and Connecticut Avenue, Washington, D.C. These signs are representative of fixed schedule lane occupancy signs in the Washington, D.C., area.

Series 2 signs presented the same regulations as Series 1 in the form of printed instructions, such as "Right Lane Only," "Through Traffic Either Lane," and "Left Lane Closed." The rules shown were those in force at the time given in the corresponding Series 1 question.

Series 3 changeable message signs presented the MUTCD-approved red X and green arrow symbols. Series 4 changeable message signs indicated lane status by the words "closed" or "open." Where necessary, printed messages were used.

[3] "Manual on Uniform Traffic Control Devices for Streets and Highways," *National Advisory Committee on Uniform Traffic Control Devices, U.S. Department of Transportation*, Washington, D.C., 1978.

[4] C. Dudek, R. Hutchinson, R. Koppa, and M. Edwards, "Human Factors Evaluation of Traffic State Descriptor Variables: Vol. 9—Human Factors Requirements for Real-Time Motorist Information Displays," Report No. FHWA-RD-78-13, *Federal Highway Administration*, Washington, D.C. Report not yet published.

Fixed schedule signs (Series 1)
NO LEFT TURN AT CANAL RD. 7:00 - 9:30 AM 4:00 - 6:30 PM MONDAY – FRIDAY EXCEPT HOLIDAYS
2 LANES 7 - 9:30 AM MONDAY – FRIDAY EXCEPT HOLIDAYS 3 LANES ALL OTHER TIMES
RIGHT LANE CLOSED 3:30 PM - 5:30 PM MON. THRU FRI.
7 - 9:30 AM MONDAY – FRIDAY EXCEPT HOLIDAYS
ALL OTHER TIMES
MUTCD mixed signs (Series 3)
(a)
(b)
LANES LEFT CENTER RIGHT
YES
YES
Texas Transportation Institute signs (Series 4)
LANES LEFT CLOSED CENTER CLOSED RIGHT OPEN
LANES LEFT OPEN CENTER OPEN RIGHT MUST TURN RIGHT
ON CHAIN BRIDGE CENTER CLOSED RIGHT OPEN
Printed message signs (Series 2)
NO LEFT TURN
THROUGH TRAFFIC EITHER LANE
LEFT LANE CLOSED
LEFT TURN TRAFFIC USE RIGHT LANE ON CHAIN BRIDGE

Figure 2.—Signs used in the lane occupancy study.

Procedure

The slides were projected on a screen 0.52 m high by 0.49 m wide (1.7 ft by 1.6 ft), 0.61 m (2 ft) in front of the subject (fig. 3). The subject viewed the sign and signaled his or her answer by pressing the "yes" or the "no" button on a console. The subject's response was indicated on the experimenter's console by coded lights; response time appeared on a digital timer. The timer started when the sign was projected on the screen and stopped when the subject responded. After each response, the experimenter reset the timer and showed the next sign when the subject said, "Ready."

Twenty-two questions were asked on each sign series. The answers to these questions and the associated response times constituted the experimental data of the study.

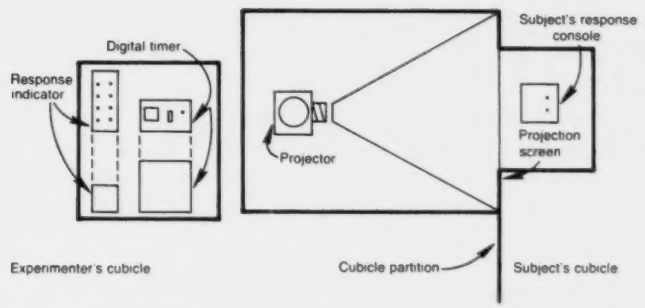


Figure 3.—Schematic diagram of the experimental setup.

Results

Table 1 presents a summary of the experimental results. The effectiveness of the tested sign series is indicated by the errors and response times. Each number in table 1 is based on 880 separate responses by the 40 subjects to the 22 questions.

Errors

An error was made when a subject's response violated the lane occupancy instructions shown on the sign. Table 1 shows that 143 errors were made on Series 1 signs; 20, 16, and 8 errors were made on the three changeable message sign series. These results indicate 1 error for every 6.15 presentations of Series 1 signs, and 1 error per 44, 55, and 110 presentations for the three changeable message sign series. The changeable message series signs elicited considerably fewer errors than the Series 1 signs. In all three comparisons between fixed schedule and changeable message series, the differences found in the number of errors would have occurred by chance less than once per 100,000 replications of the study (Binomial Expansion Test). The fixed schedule sign average of 1 error per 6.15 trials represents a dangerous performance that could lead to an accident.

Among the changeable message signs, the printed message signs (Series 2) produced more errors than the MUTCD-approved (Series 3) or Texas A&M (Series 4) signs. But none of the differences among the changeable message series reached a 0.05 level of statistical significance.

Table 1.—Lane occupancy study results

	Sign series			
	1	2	3	4
Total errors	143	20	16	8
Mean errors per question	0.163	0.023	0.018	0.009
Mean trials per error	6.15	44.0	55.0	110.0
Mean response times (seconds)	5.49	2.44	1.84	1.81

Response Times

On the average, subjects took more than twice as long to respond to Series 1 signs than to respond to any of the other sign series (table 1). The superiority in response time of each of the changeable message series over the fixed schedule sign series would occur by chance less than once per million replications of the study. The subjects' response times to the changeable message series may be regarded as minimal; a driver on the road would require more time to make a judgment than did the subjects performing in the laboratory.

In Series 1, 16 of 22 questions had 25 percent or more subject errors or dangerously delayed responses (longer than 10 seconds). None of the responses for the

changeable message sign series was as much as 25 percent faulty. Differences between the Series 1 signs and any one of the changeable message series signs would occur by chance less than once per million replications of the study.

Drivers' Appraisals of the Signs

After completing the lane occupancy questions, subjects were asked to rank and rate each sign series. Sign Series 3 was divided into a and b, shown in figure 2. To help them rank the sign series, subjects were shown examples of the signs that they had responded to earlier in the experiment. These signs were shown as often as requested by the subject. Ranking data were obtained from 21 subjects.[5]

The results in table 2 show that the fixed schedule signs were ranked lowest by 19 of the 21 subjects. The mean rankings were obtained by weighting each rank by its associated frequency and dividing the total by the 21 responses. The Series 1 fixed schedule sign had an average ranking of 4.71 on a 5-point scale where 1 was high and 5 was low. Printed messages were ranked lowest of the changeable message types.

Table 2.—Rankings of sign series

Sign series	Rank					Mean rank
	1	2	3	4	5	
1	1	0	1	0	19	4.71
2	5	0	4	12	0	3.09
3a	8	5	5	3	0	2.14
3b[1]	1	11	4	3	1	2.60
4[1]	6	4	7	3	0	2.36

[1] In one instance there was a tie between second and third place. Both signs were ranked 2.5.

Rating results conform with the rankings (fig. 4). To rate the series, each subject marked his or her assessment on a printed scale. The lines shown in figure 4 indicate the range of subjects' ratings from the first to the third quartile. The dot on the line shows the median rating.

[5] Data are not available for the remaining 19 of the 40 subjects.

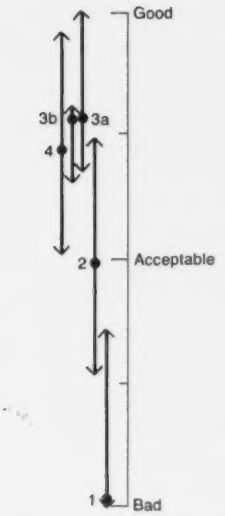


Figure 4.—Ratings of sign series—the median rating is shown by a dot; the 25th and 75th percentile ratings are shown by arrow extremes for each rated series.

The median rating of changeable message sign Series 3a, 3b, and 4 fall between "acceptable" and "good." The median rating of the printed message Series 2 signs is slightly below "acceptable." Series 1 fixed schedule signs are rated close to "bad."

Subjects spontaneously expressed difficulties with the fixed schedule signs. Typical reactions were as follows:

"The first kind, that is the kind they have in D.C., I can't go there. I get so confused with the lanes. I have somebody come with me." (Subject 38)

"It's too mind boggling to figure the times and the exceptions." (Subject 36)

"It takes too long to read." (Subject 33)

"The changeable messages are so much easier to read than these signs." (Subject 32)

"These are the ones that just throw me." (Subject 26)

"I just can't react that fast to a written sign." (Subject 25)

"You don't have time to read a whole sign. That's why symbols are better." (Subject 24)

"Bad. I hate those signs." (Subject 21)

"Should be discontinued. You have to stop to understand it." (Subject 18)

Discussion

These laboratory results demonstrate the advantages of changeable message signs over fixed schedule lane occupancy signs. Changeable message signs give more accurate responses, are more quickly interpreted, and are preferred by drivers.

Among changeable message signs, the Series 3 MUTCD-approved red X and green arrows and the Series 4 Texas A&M design were more effective than the Series 2 printed message signs. Series 3 and 4 signs had fewer total errors, shorter response times, and were more highly ranked than the Series 2 signs. The Texas A&M sign format has the additional advantage of being able to show incident messages such as "right lane closed," "closed accident," and "must turn right," in addition to the "open" and "closed" lane occupancy messages tested in this study. To reduce installation costs, it would be advantageous to display occupancy information in the highway shoulder-mounted position. On-the-road field studies would be required to test the practicality of this technique.

These lane occupancy sign studies raise questions on the use of fixed schedule occupancy signs. Incorrect judgments were made 7 times more often on the fixed schedule signs than on any of the changeable message series—1 error per 6.15 presentations. Subjects took more than twice as long to interpret fixed schedule signs than to interpret signs in the other series. Twenty-five percent or more wrong responses or dangerously delayed responses (longer than 10 seconds) were given on 73 percent of the fixed schedule signs. Drivers mentioned difficulties they had experienced with the fixed schedule signs now on the road; these signs were rated "bad" by more than half of the subjects. The effectiveness of fixed schedule lane occupancy signs should be examined further in field studies. Possibly such signs need to be replaced by low-cost changeable message signs, yet to be developed.

Donald A. Gordon is a research psychologist in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been involved in research on the driving process and the development of improved signs and roadway delineations. He has published articles on topics such as perception in vehicular guidance, experimental isolation of the driver's visual input, and the contribution of psychology to the traffic flow theory. Lately, he has been concerned with the development of improved highway guide signs.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Division. The reports are available from the address noted at the end of each description.

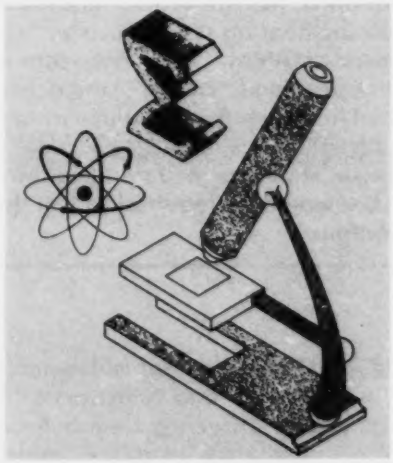
Technical Guidelines for Expansive Soils in Highway Subgrades, Report No. FHWA-RD-79-51

by FHWA Materials Division

Volume change of expansive soil subgrades resulting from moisture variations causes highway damage estimated at $1.7 billion annually. This report concludes a 4 1/2-year study to evaluate methods for predicting and minimizing detrimental volume change of expansive soils in highway subgrades. The report presents technical guidelines for: Locating potentially expansive soil areas using occurrence and distribution maps; exploring and sampling expansive soils in the field; identifying and classifying potentially expansive soils using index and soil suction properties; testing expansive soils and predicting volume change; selecting appropriate treatment methods; and presenting design, construction, and maintenance recommendations for new and existing highways.

Appendixes to the report describe the soil suction test procedure, a standard procedure for odometer swell tests, a procedure for calculating the potential vertical rise (PVR), and standards for field monitoring data. A bibliography of alternative treatments is included.

The report is available from the Materials Division, HRS-21, Federal Highway Administration, Washington, D.C. 20590.



Power Plant Bottom Ash in Black Base and Bituminous Surfacing, Executive Summary (Report No. FHWA-RD-79-72) and Users Manual (Report No. FHWA-RD-78-148)

by FHWA Materials Division

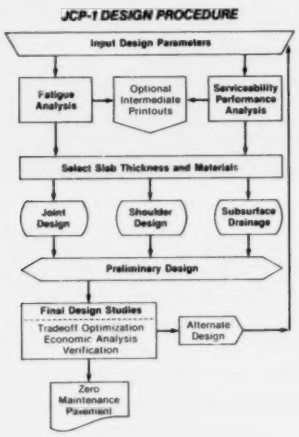
Because of dwindling supplies of natural aggregates in some areas of the United States and increasing shipping costs, the use of bottom ash and other waste substitutes in bituminous paving mixtures is being explored. The increasing use of coal is producing a great amount of bottom ash. Acceptable use of this waste material for highway construction would alleviate the economic and environmental problems confronting power plants having large amounts of this waste. These reports present the findings of an investigation of the use of bottom

ash as a partial replacement or extender of natural aggregate in bituminous highway paving mixtures. The ash samples subjected to laboratory screening tests generally met specification requirements for an aggregate in bituminous mixtures.

Both dry and wet bottom varieties of the ash are described in the reports. Results of Marshall testing of bituminous mixtures containing varying proportions of ash and natural aggregate are presented. These results are compared with Marshall testing of bituminous mixtures containing only natural aggregate or bottom ash.

The reports are available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.

Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume I (Report No. FHWA-RD-77-111) and Volume II (Report No. FHWA-RD-77-112)



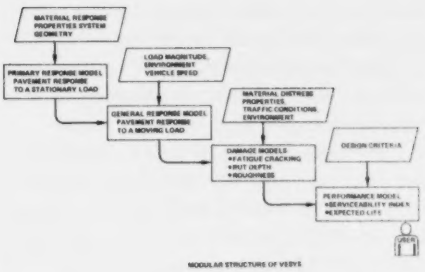
by FHWA Structures and Applied Mechanics Division

By definition, a zero-maintenance pavement does not require structural maintenance such as patching, crack or joint filling, slab replacement, grinding, or overlay during the initial design life.

Volume I, **Development of Design Procedures,** documents procedures for the structural design of plain jointed concrete highway pavements to provide zero-maintenance service under heavy traffic over a selected design period. The design procedures are based on nationwide field studies, long term performance data of inservice pavements, comprehensive mechanistic analyses, and laboratory studies. Both a serviceability-performance analysis and a concrete fatigue damage analysis are used to develop procedures for the structural design of the concrete slab, subbase, shoulders, joints, and subsurface drainage. Stresses in the slab resulting from traffic loading and from thermal gradients are considered in the analysis through use of a finite element model. A new serviceability-performance design model was derived for plain jointed concrete pavements based on long term performance data on sections of the American Association of State Highway Officials Test Road which have continued in service.

Volume II, **Design Manual,** is an engineering guide for the design of heavily trafficked plain jointed concrete pavements to provide zero-maintenance performance over a given period. Recommendations are made for obtaining necessary structural design inputs. A detailed design example is provided, which includes sensitivity and incremental cost analyses of the major design factors. Input and output listings are included for the computer program (JCP-1) written for the structural design of the pavement.

These reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 289394 and PB 289395).



MODULAR STRUCTURE OF VESYS

Effects on Flexible Highways of Increased Legal Vehicle Weights Using VESYS IIM, Report No. FHWA-RD-77-116

by FHWA Structures and Applied Mechanics Division

This report describes a study that established a predictive model for relating flexible pavement life-cycle costs for 20 years of acceptable service life to incremental increases in allowable axle load limits.

VESYS IIM, a computerized routine developed by FHWA for the analysis and design of flexible pavements, accepts input data describing the pavement structure, traffic loadings, temperatures, and seasonal material characterizations that reflect the pavement's environment. The results are predictions of fatigue cracking, rut depth, slope variance, the Present Serviceability Index, and expected life as a function of time in terms of truck traffic and axle load distribution.

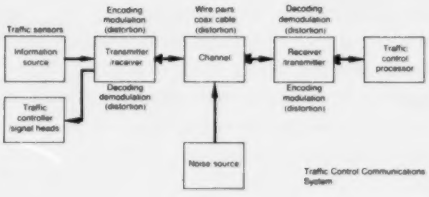
The effects of four single-axle load levels—80, 89, 98, and 107 kN (18, 20, 22, and 24 kips)—and two levels of

truck traffic volume were evaluated for two pavement thicknesses and four environmental zones—wet/freeze, dry/freeze, wet/no freeze, and dry/no freeze. When calculated pavement failures for the assumed design occurred in less than 20 years, trial overlays were introduced into the analysis and new life expectancies were predicted until a total pavement life of at least 20 years was attained. The costs of initial and overlay construction were estimated and related to the various axle load limits for 20 years of pavement service.

This analytical procedure was verified by comparisons of predicted and actual distress in 12 flexible pavement test sections throughout the United States. Performance and cost data are presented in the report for examples of both new and existing typical flexible pavements. The procedure and quantitative relationships are sufficiently accurate for useful pavement life analyses until more comprehensive procedures are available.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 289697).

A Communications Trade Off Study for Computerized Traffic Control, Volume 1 (Report No. FHWA-RD-78-85) and Volume 2 (Report No. FHWA-RD-78-86)



by FHWA Traffic Systems Division

This report describes communication techniques that will reduce costs of future computerized urban traffic control systems. The information is presented in the form of reference material, cost computation procedures, and a communication subsystem specification example.

Volume 1, **Final Report,** discusses the principal functions of a computerized traffic control system, describes the types of data that must be transmitted, reviews the available communication methods, and shows how to estimate the transmission rate requirements for typical systems. It also discusses factors affecting communication costs and describes procedures for computing costs and utility measures. Volume 2, **Appendixes,** provides additional detailed technical information.

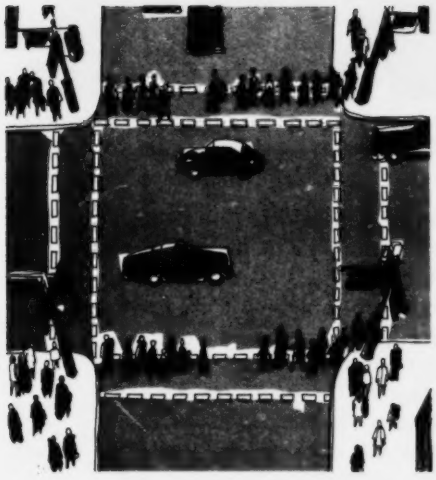
The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 298843 and PB 298844).

Evaluation of Highway Advisory Radio in the I-35W Traffic Management Network, Report No. FHWA-RD-79-33

by FHWA Traffic Systems Division

This report presents the Minnesota Department of Transportation's experiences with and assessment of highway advisory radio (HAR) in a traffic surveillance and control network. The report describes HAR system hardware, its integration with traffic management operations, and its effectiveness as a traffic control device and covers the installation and operation of HAR for a 1-year period. An analysis of major system elements, message characteristics, motorist use and acceptance, and various system costs was made from the evaluation data.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 297142).



A Pedestrian Planning Procedures Manual, Volume I (Report No. FHWA-RD-79-45), Volume II (Report No. FHWA-RD-79-46), and Volume III (Report No. FHWA-RD-79-47)

by FHWA Environmental Division

This three-volume manual identifies relevant data, procedures, and criteria for planning and evaluating comprehensive pedestrian systems and individual facilities.

Volume I, **Overview,** provides a general background and introduction to the pedestrian planning process. Volume II, **Procedures,** describes operational methods which may be used for estimating pedestrian demands and evaluating pedestrian networks. Volume III, **Technical Supplement,** explains the derivation of the information provided in Volume II and presents detailed data and methodologies for various tasks. Illustrative examples are included.

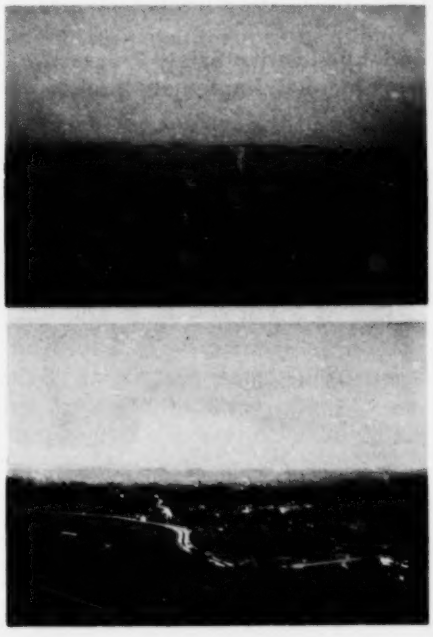
The reports are available from the Environmental Division, HRS-41, Federal Highway Administration, Washington, D.C. 20590.

Highway Air Quality Impact Appraisals, Volume I (Report No. FHWA-RD-78-99) and Volume II (Report No. FHWA-RD-78-100)

by FHWA Environmental Division

Although much is known about air pollution, it has not been a major consideration in transportation planning. These reports guide transportation planners and engineers in performing air quality analyses and in incorporating air quality considerations into the highway planning and design process.

Volume I, **Introduction to Air Quality Analysis,** presents information for analyzing local or regional air quality impacts. It is useful as a general reference, particularly in the areas of emissions, atmospheric processes, and air quality models.



Volume II, **Guidance for Highway Planners and Engineers,** discusses air quality issues that arise in evaluating land use plans, transportation policies, and facility and operation plans at both regional and subregional levels. It familiarizes the highway developer with issues that must be considered in determining the appropriate analysis method. Laws that require air quality evaluations are reviewed.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 293798 and PB 293799).

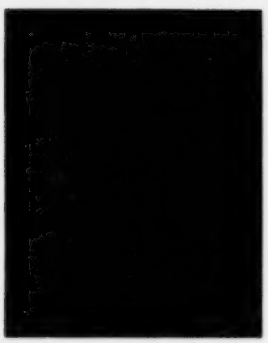
Implementation/User Items "how-to-do-it"

The following are brief descriptions of selected items which have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies.

U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division (HDV-20)
Washington, D.C. 20590

Items available from the Implementation Division can be obtained by including a self-addressed mailing label with the request.

Roadway Lighting Handbook, Implementation Package 78-15

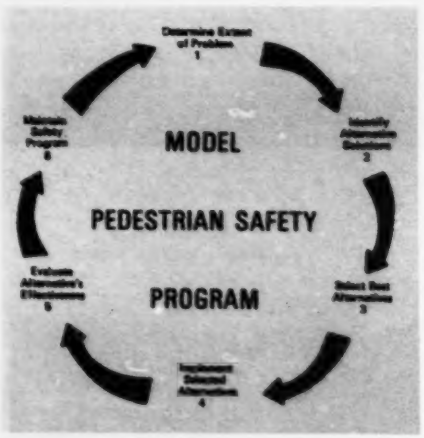


by FHWA Implementation Division

This comprehensive, up-to-date handbook on public roadway lighting is intended as a guide in planning, designing, operating, and maintaining roadway lighting systems. It may be used as a reference document by engineers and public officials or as a textbook for formal education and training in roadway lighting. The overall process of designing and operating a roadway lighting system consists of several well-defined steps, which are identified as chapters in the handbook. International source material is incorporated.

The handbook is available for $4.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-003-00339-5).

Model Pedestrian Safety Program, Users Manual, Implementation Package 78-6



by FHWA Implementation Division

Local governments and the highway safety community are recognizing the magnitude and severity of the pedestrian accident problem. It is important to disseminate the latest information on accident reduction methods in a format readily usable by local officials. This manual provides comprehensive information on pedestrian accident reduction techniques and a step-by-step description of a systematic pedestrian safety program. The manual is intended to assist localities in establishing programs for planning, implementing, and evaluating pedestrian safety improvements.

A six-step process for a pedestrian safety program is identified in the manual—Step 1: Determine the extent of the pedestrian safety problem; Step 2: Identify alternative solutions; Step 3: Select the best alternatives (benefit-cost analysis); Step 4: Implement selected alternatives; Step 5: Evaluate the effectiveness of the implemented alternatives; and Step 6: Maintain the pedestrian safety program. Each step is discussed in detail. In addition, several appendixes provide more detailed technical data and a glossary of terms used in the text.

Limited copies of the manual are available from the Implementation Division.

Optimizing Maintenance Activities, Sixth Report, Sign Maintenance, Report No. FHWA-TS-78-223

by FHWA Implementation Division

This report, the sixth in a special series on highway maintenance, summarizes the results of a sign maintenance value engineering study in Arkansas, Florida, and Kentucky. Included in the report are recommendations in the areas of proliferation of signs, traffic sign supports, centralized versus decentralized sign shops, and equipment and procedures. Purchasing procedures and sign lighting are also discussed, as is the use of a thin overlay to repair damaged signs.

Implementation of the study results will have different effects in each of the three study States depending upon current maintenance practices. Modifying existing procedures should result in substantial cost savings.

Limited copies of the report are available from the Implementation Division.

Field Maintenance Manual for Georgia Counties' Local Roads and Streets, Report No. FHWA-TS-79-218

by FHWA Implementation Division

This manual was prepared to assist maintenance personnel in Georgia county highway departments in identifying maintenance problems, causes of these problems, and corrective measures. Inventories of deficiencies and available equipment and personnel were taken in several Georgia counties. Specific areas studied include pavement, shoulder, drainage, bridge structure, and grounds maintenance.

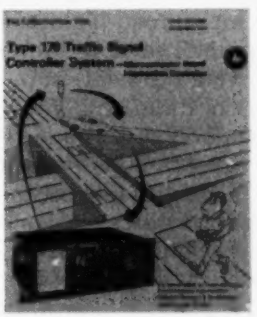
Although limited to highway maintenance problems in Georgia, the manual and corrective measures identified have nationwide application. The manual should be of interest to materials, maintenance, and pavement design engineers, and particularly maintenance personnel from State, city, and county highway agencies.

Limited copies of the manual are available from the Implementation Division.

Type 170 Traffic Signal Controller System—Microcomputer Based Intersection Controller, Report No. FHWA-TS-78-228

by FHWA Implementation Division

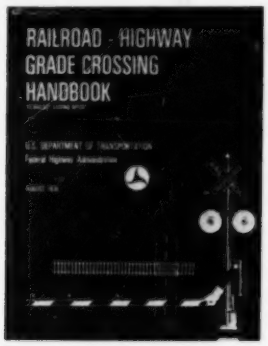
Traffic demands at some urban diamond interchanges have grown at increasing rates. This growth has increased traffic on the ramps of the diamond and the adjacent street system and caused a low level of service during rush periods. Considerable work has been done on real-time control of diamond interchanges in an attempt to optimize traffic flow throughout the interchange and retain smooth flow on adjacent corridors.



This report compares the 170 microcomputer-based controller to the minicomputer equipment and the hardwired equipment for diamond interchange control to show the flexibility and low cost of the controller. User orientation of the controller is explained and software considerations, such as man/machine interface, are described. The report concludes with a recommendation that the 170 controller be considered as a standard controller for all traffic signal control systems. The recommendations apply to most signalized intersections and ramps.

Limited copies of the report are available from the Implementation Division.

Railroad-Highway Grade Crossing Handbook, Report No. FHWA-TS-78-214



by FHWA Implementation Division

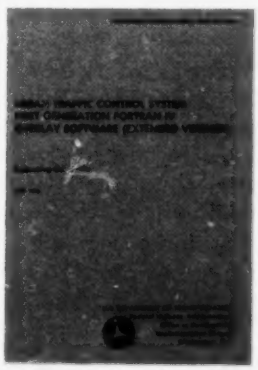
This handbook is a summary of applicable concepts, technology, and practice for railroad-highway grade crossings and provides information on improving the safety and efficiency of these crossings. The basic components of grade crossings are described by reporting on existing grade crossing technology. The handbook describes conditions and requirements at crossings, presents elements of crossing systems, reports on improvements to railroad-highway grade crossings, and aids in the application and understanding of new technology.

The handbook is available for $4.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-005-00027-5).

Urban Traffic Control System First Generation FORTRAN IV Overlay Software (Extended Version), Executive Summary, Report No. FHWA-TS-79-222

by FHWA Implementation Division

This report describes the control software developed in Charlotte, N.C., for FHWA. The software is used in traffic signal systems that are controlled by a digital computer. The software has been successfully operating for over 1 year and has been specified for several other control systems across the Nation.



This software package replaces earlier releases of the first generation software and contains more functional capabilities and features to reduce time and costs in implementing and operating the software in a typical computerized traffic control system.

Limited copies of the report are available from the Implementation Division.

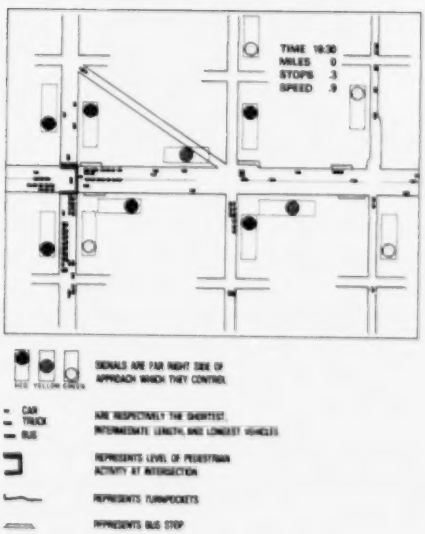
NETSIM

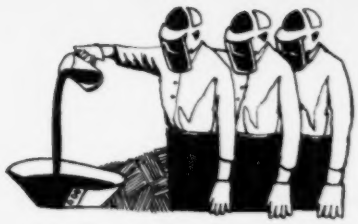
by FHWA Implementation Division

Traffic engineers have long needed an aid for evaluating the costs and benefits of alternative traffic control methods. Computer simulation modeling enables the engineer to inexpensively select the best alternative before funding its implementation. The Network Simulation Model (NETSIM) was developed by FHWA to fill the traffic engineer's need and has been used successfully throughout the United States in the last few years.

This 16 mm color/sound film demonstrates the capabilities of the NETSIM model and shows that traffic simulation can reliably predict the effects of transportation engineering changes on traffic flow in an urban street network.

The movie is available on loan from the National Highway Institute.





New Research in Progress

The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. Space limitation precludes publishing a complete list. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1V: Roadside Safety Hardware for Nonfreeway Facilities

Title: Multiple Service Level Highway Bridge Railings—Selection Procedures. (FCP No. 51V2113)
Objective: Develop bridge railing systems for a number of service levels. Determine the number of service levels needed based on the total costs of these railing systems. Refine multiple service level approach procedures accordingly. Develop an upgrading strategy. Assess the legal implications. Prepare a users manual.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: December 1980
Estimated Cost: $200,000 (NCHRP)

FCP Project 1W: Skid Resistance Inventory Procedures and Equipment

Title: Pavement Surface Texture—Significance and Measurement. (FCP No. 31W2022)
Objective: Develop, evaluate, and demonstrate measurement methods for pavement surface texture and demonstrate effects of texture on skid resistance and on skid resistance-speed gradients. Establish secondary effects of texture and their significance. Develop empirical relationships between pavement texture and specific variables.
Performing Organization: Pennsylvania State University, University Park, Pa. 16802
Expected Completion Date: April 1982
Estimated Cost: $244,000 (FHWA Administrative Contract)

Title: Deployment of a Digital Road Profilometer. (FCP No. 41W3112)
Objective: Prepare specifications for profilometer. Adapt software. Make procedural changes. Continue to provide road surface measurements.
Performing Organization: University of Texas, Austin, Tex. 78701
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1981
Estimated Cost: $170,000 (HP&R)

FCP Project 1Y: Traffic Management in Construction and Maintenance Zones

Title: Determination of Driver Needs in Work Zones. (FCP No. 31Y1052)
Objective: Analyze driver tasks associated with approaching and traversing work zones. Determine and establish priorities for types of information needed by driver and how to convey information to driver. Determine needs for improvement of present system of traffic control devices.
Performing Organization: Institute for Research, State College, Pa. 16801
Expected Completion Date: December 1980
Estimated Cost: $311,000 (FHWA Administrative Contract)

FCP Category 2—Reduction of Traffic Congestion, and Improved Operational Efficiency

FCP Project 2C: Requirements for Alternate Routing to Distribute Traffic Between and Around Cities

Title: IMIS Phase 3—Development of Detailed Design and PS&E. (FCP No. 32C3042)
Objective: Design integrated motorist information system (IMIS) to be demonstrated on Northern Long Island Corridor. Include traffic surveillance, radar metering, incident management, route diversion, arterial signal control, and motorist aid in emergencies.
Performing Organization: Sperry Systems Management Division, Great Neck, N.Y. 11020
Expected Completion Date: August 1981
Estimated Cost: $2,348,000 (FHWA Administrative Contract)

FCP Category 3—Environmental Considerations in Highway Design, Location, Construction, and Operation

FCP Project 3E: Reduction of Environmental Hazards to Water Resources Due to the Highway System

Title: Efficiency of Erosion Control Practices. (FCP No. 43E2224)
Objective: Evaluate the effectiveness of current erosion and sediment control practices. Determine optimum levels of erosion and

sediment control using design, construction, and maintenance of control procedures.
Performing Organization: Virginia Highway Research Council, Charlottesville, Va. 22903
Funding Agency: Virginia Department of Highways and Transportation
Expected Completion Date: June 1981
Estimated Cost: $161,000 (HP&R)

Title: **Assimilative Capabilities of Highway Storm Water Runoff Retention Ponds. (FCP No. 43E3332)**
Objective: Evaluate a retention basin based upon the first 25.4 mm (1 in) of highway storm water runoff. Determine its effectiveness as a mitigation measure. Determine reduction in pollutants, effects to ground water, and uptake by in-basin vegetation.
Performing Organization: U.S. Geological Survey, Orlando, Fla. 32800
Funding Agency: Florida Department of Transportation
Expected Completion Date: June 1982
Estimated Cost: $260,000 (HP&R)

FCP Project 3F: Pollution Reduction and Environmental Enhancement

Title: **Evaluation of Highway Air Pollution Dispersion Models. (FCP No. 53F3502)**
Objective: Develop methodology for evaluating highway dispersion model performance. Document a composite highway aerometric data base. Perform evaluation of selected line source dispersion models using above methodology and data.
Performing Organization: Stanford Research Institute International, Menlo Park, Calif. 94025
Expected Completion Date: June 1981
Estimated Cost: $200,000 (NCHRP)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4G: Substitute and Improved Materials to Effect Materials and Energy Conservation in Highways

Title: **Evaluation of Sulfur-Asphalt Paving Mixtures in Ohio. (FCP No. 44G1302)**
Objective: Conduct laboratory and field performance evaluation of sulfur-asphalt paving mixtures related to experimental construction projects in Ohio.
Performing Organization: Ohio State University, Columbus, Ohio 43216
Funding Agency: Ohio Department of Transportation
Expected Completion Date: March 1982
Estimated Cost: $146,000 (HP&R)

FCP Project 4K: Cost Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments

Title: **Use of Improved Structural Materials in Marine Piling. (FCP No. 34K3073)**
Objective: Evaluate and compare the commercial manufacture, handling, driving, and field performance of marine piling made from polymer concrete, polymer-impregnated concrete, internally sealed concrete, latex-modified concrete, and concrete containing epoxy-coated rebars. Install the piles in a marine environment and compare with conventional prestressed concrete piles.
Performing Organization: Oregon Department of Transportation, Salem, Oreg. 97310
Expected Completion Date: October 1982
Estimated Cost: $149,000 (FHWA Administrative Contract)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5A: Improved Protection Against Natural Hazards of Earthquake and Wind

Title: **Wind Instrumentation of Mississippi River Bridge (Luling, La.). (FCP No. 45A1112)**
Objective: Evaluate wind induced vibrations of the stays, vortex induced motion of the bridge deck, and wind climate and response of the bridge under extreme wind conditions.
Performing Organization: Tulane University, New Orleans, La. 70118
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: October 1981
Estimated Cost: $207,000 (HP&R)

FCP Project 5B: Tunneling Technology for Future Highways

Title: **Improved Structural Design for Frozen Support Systems. (FCP No. 35B1432)**
Objective: Develop improved structural design criteria and guidelines for frozen ground support systems. Review and evaluate current structural design technology. Determine where improvements can be made. Conduct experimental testing programs. Develop improved design criteria and structural concepts and write comprehensive guidelines for support systems.

*U.S. Government Printing Office: 1979—620-103/1

Performing Organization: Law Engineering Testing, McLean, Va. 22101
Expected Completion Date: September 1980
Estimated Cost: $127,000 (FHWA Administrative Contract)

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: An Integrated Pavement Data Management and Feedback System (PAMS). (FCP No. 45D1322)
Objective: Determine the feasibility of developing a pavement data management system to develop this system.
Performing Organization: Louisiana Department of Transportation and Development, Baton Rouge, La. 70804
Expected Completion Date: December 1982
Estimated Cost: $162,000 (HP&R)

Title: Evaluation of Fabric Underseals. (FCP No. 45D2602)
Objective: Evaluate the performance of fabrics to establish realistic specification limits. Determine the types of distress, if any, that fabrics can economically correct.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843.
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: June 1983
Estimated Cost: $190,000 (HP&R)

Title: Performance, Reliability, and Durability of State of the Art Weigh-In-Motion Systems for Highway Vehicles. (FCP No. 45D3214)
Objective: Evaluate system performance, reliability, and durability of present state of the art weigh-in-motion bridges and systems for acquisition of highway statistical traffic data (phase A) and for truck overweight screening (phase 3).
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: June 1981
Estimated Cost: $208,000 (HP&R)

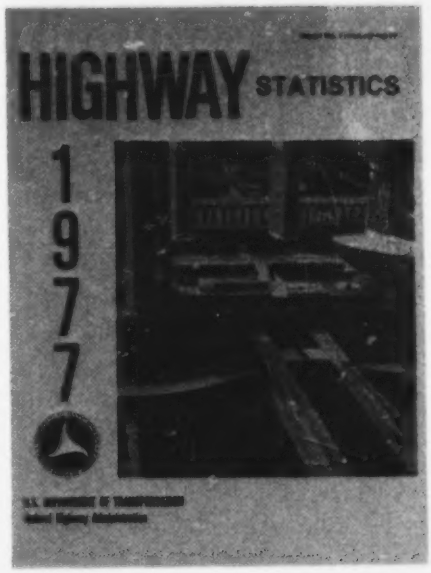
FCP Project 5J: Rigid Pavement Systems Design

Title: Mechanistic Design of Rigid Pavements. (FCP No. 35J2012)
Objective: Develop a new, practical, and implemental procedure for the structural design of rigid pavements (except prestressed concrete). Include tasks on literature and design methodology review, identification of distress mechanisms, development of mathematical models and computer programs, and field verification.
Performing Organization: Resource International, Worthington, Ohio 43085
Expected Completion Date: April 1982
Estimated Cost: $245,000 (FHWA Administrative Contract)

FCP Project 5L: Safe Life Design for Bridges

Title: Fatigue Testing of Weathered A588 Beams. (FCP No. 45L1082)
Objective: Determine the fatigue behavior of full size beams fabricated from A588 structural steel with a rust coat corresponding to 3 years of weather exposure. Obtain some 7-year weathering of small-scale tensile specimens.
Performing Organization: University of Maryland, College Park, Md. 20742
Funding Agency: Maryland State Highway Administration
Expected Completion Date: August 1984
Estimated Cost: $144,000 (HP&R)

New Publication



Highway Statistics, 1977, the 33rd publication in the annual series, presents statistical and analytical tables of general interest on motor fuel, motor vehicles, driver licensing, highway-user taxation, State and local highway financing, road and street mileage, and Federal-aid for highways. Also reported are 1976 highway finance data for municipalities, counties, townships, and other units of local government. A listing of the data is given in the table of contents and a brief discussion is given in the text accompanying each section.

The publication may be purchased for $5.25 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 00142-0). It is also available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 for $5.25 (Report No. FHWA-HP-HS-77) and in microfiche copy for $3 (PB No. 296097).

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Is Delineation Needed?

Development and Testing of Advanced Control Strategies in the Urban Traffic Control System

The Evaluation of a Noncontact Profiling System Using the Acoustic Probe

Laboratory Studies of Lane Occupancy Signs

public roads

A JOURNAL OF HIGHWAY RESEARCH AND DEVELOPMENT

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U.S. Department of Transportation — Federal Highway Administration

March 1980 — Vol. 43, No. 4

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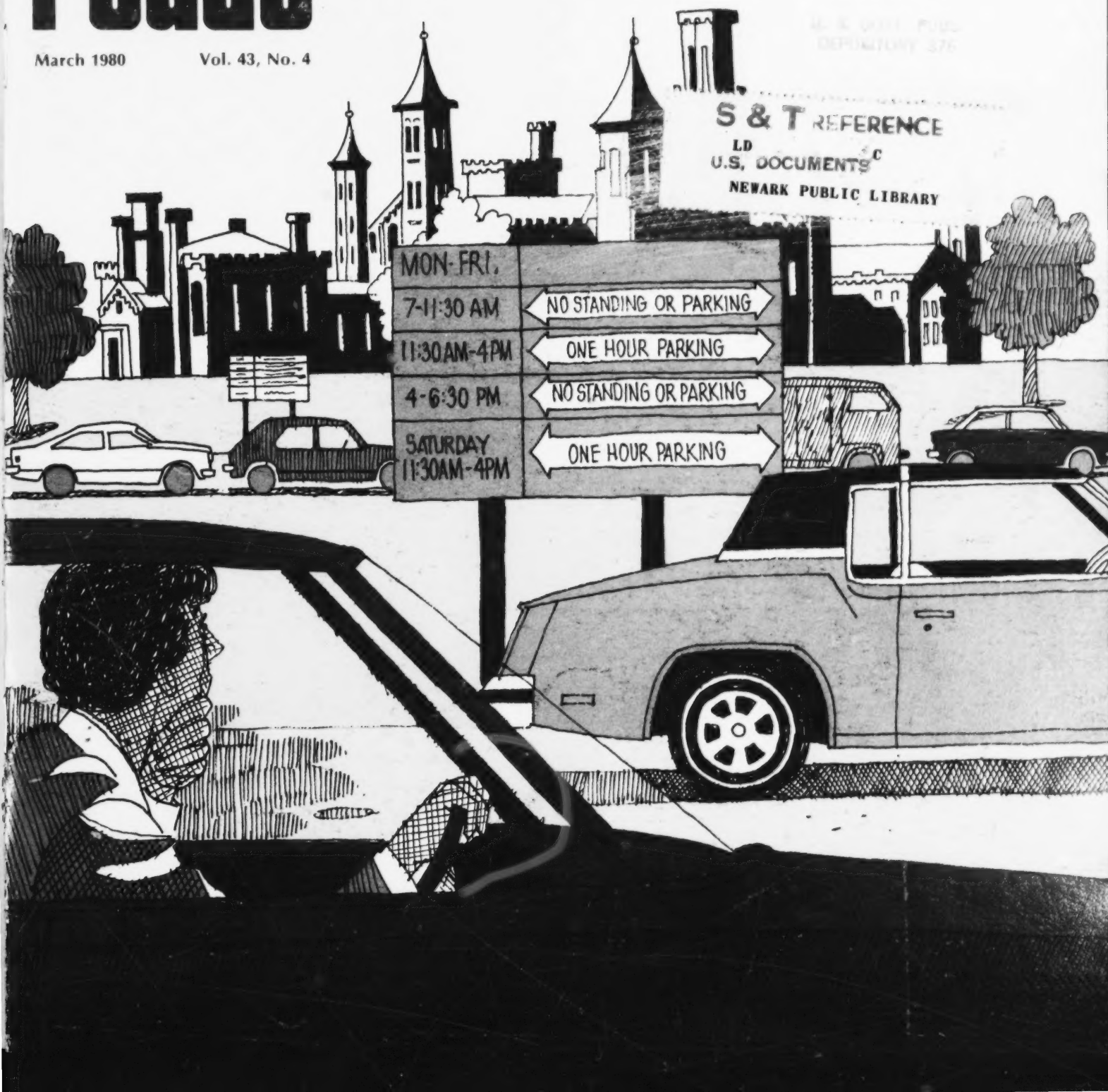

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March 1980 Vol. 43, No. 4

COVER:
Artist's concept of confusing parking signs.

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The Development of Improved Parking Signs

by

Donald A. Gordon[1]

Introduction

"Laboratory Studies of Lane Occupancy Signs," by Donald A. Gordon, in the December 1979 issue of *Public Roads* describes Federal Highway Administration research on laboratory testing of drivers' decisions on lane occupancy.

This article summarizes the results of laboratory studies on parking signs. Subjects indicated whether displayed parking signs permitted parking or standing at designated times. Their responses indicated the designs that drivers most accurately and rapidly understood and the designs that caused difficulty for drivers.

Conventional Parking Signs

The Manual on Uniform Traffic Control Devices (MUTCD) permits the parking sign types shown in figure 1. *(1)*[2] Sign types a and c display prohibitions in red letters on a white background; sign types b and d display permissive information in green letters on a white background. The restriction and duration of parking are emphasized in each sign type. Although these signs are effective, difficulties arise when the regulations are applied to complicated parking and standing schedules. *(2)* Double- and triple-paneled conventional signs can be difficult to interpret (fig. 2).

The driver may have to search through two or three panels to find information he or she needs: The top panel may concern standing, whereas the driver may want to park. The driver may have to apply one or more of the following rules: (1) If parking or standing is not prohibited, it is allowed; (2) if the sign explicitly prohibits standing, it implicitly prohibits parking; (3) if the sign prohibits only parking, it allows standing; and (4) if only standing is allowed, parking is prohibited. In some instances the information is intrinsically confusing. The top panel may cover Monday through Friday, whereas the bottom panel may cover Monday through Saturday. It is also confusing if too much information is presented on the sign: Parking and standing may be regulated to the left and right of the sign at different times of day and different days of the week.

[1]This article is a condensation of the report "Regulatory Signs for Lane Occupancy and Parking," by D. A. Gordon, Report No. FHWA-RD-78-89, *Federal Highway Administration*, Washington, D.C., June 1978.

[2]Italic numbers in parentheses identify references on page 133.

Figure 1.—MUTCD-approved parking sign designs.

Time-Referenced Parking Signs

It was hypothesized that the driver could read multipaneled parking signs more easily if the information was organized by time. To interpret a parking sign, a driver must search for the regulations applying at a particular time. If regulation times were prominently displayed on the sign, the driver might be able to make a rapid decision.

Time-referenced sign types were developed and tested in a preliminary study. The type giving the fewest errors was selected from three time-referenced designs that differed in format. This design was evaluated in a later phase of the study.

Comparison of Signs

The selected time-referenced sign type was compared with conventional parking signs and changeable message signs (figs. 2–4). The conventional series reproduced actual Washington, D.C., parking signs and included two single-paneled signs, two double-paneled signs, and two triple-paneled signs. The time-referenced series used the sign type shown most effective in the previous trials and presented the same parking regulations as the conventional series. The changeable message series presented the regulations in force at the time stated in the question. For example, if the question read "Can you park to the left of the sign at 8:30 a.m. on Wednesday?" the changeable message sign showed the restrictions applying at 8:30 a.m. on Wednesday. Although changeable message signs are too costly for actual use, they were included in the study to show driver performance when parking information is presented most simply. Changeable message signs showed regulations right and left of the sign even though the question involved parking or standing on only one side. For all three sign series, prohibitory information was printed red on a white background; permissive information was black on a white background.

Subjects and Procedure

The 48 subjects participating in the study included a range of ages, education, and driving experience (table 1). All of the subjects had valid driving permits and were paid for their participation.

Eighteen questions were asked on the conventional series and on the time-referenced series, three questions on each sign used. The same 18 questions were asked on the changeable message series, but 15 rather than 6 types of signs were required to display the information outlined in the questions. The within-series order of sign testing was random, but each series was completed before a new series was shown. Subjects responded "yes" (parking allowed) or "no" (parking prohibited). Counterbalancing insured that no particular sign series was favored by the presentation order.

Results

Errors and response times are shown in table 2. On the average, the conventional series gave one error per 3.74 trials, the time-referenced series one error per 4.96 trials, and the changeable message series one error per 6.5 trials. The correction for

Figure 2.—Conventional series signs.

ONE HOUR PARKING 7AM-6:30PM MONDAY-FRIDAY
NO STANDING 7-11:30AM 4-6:30PM MONDAY-FRIDAY
NO STANDING 7-11:30AM 4-6:30PM MONDAY-FRIDAY
ONE HOUR PARKING 11:30AM-4PM MONDAY-SATURDAY
NO STANDING 7-9:30AM 4-6:30PM
NO PARKING 9:30AM-4PM
NO STANDING 7-11:30AM
NO PARKING ANYTIME
ONE HOUR PARKING 11:30AM-6:30PM MONDAY-SATURDAY
NO PARKING ENTRANCE
NO PARKING 7-9:30AM
ONE HOUR PARKING 9:30AM-6:30PM

chance accounts for the only two possible responses—either a "yes" or a "no"—to the question. On the average, a subject would get 50 percent of the questions correct by chance (guessing) alone. The correction for chance is an indication of the subjects' true understanding of a sign series. These percentages were found using the following formula:

$$\text{Corrected percent} = \frac{100\ (R - W)}{48}$$

Where,
R = Number of subjects giving right answer.
W = Number of subjects giving wrong answer.
48 = Number of subjects.

Table 2 shows that there were fewer errors per question for time-referenced signs than for conventional signs. The difference was statistically significant at the .05 t-test level. Other differences in error scores were not statistically significant.

Table 1.—Characteristics of 48 subjects

Characteristics	Number of subjects
Sex:	
Male	31
Female	17
Years of age:	
Less than 20	13
20–29	24
30–39	6
40–49	2
Over 50	3
Education:	
Completed elementary school	6
High school graduate	16
College student	16
College graduate	10
Years of driving:	
Less than 1	5
1–9	37
10–19	3
20–29	1
Over 30	2

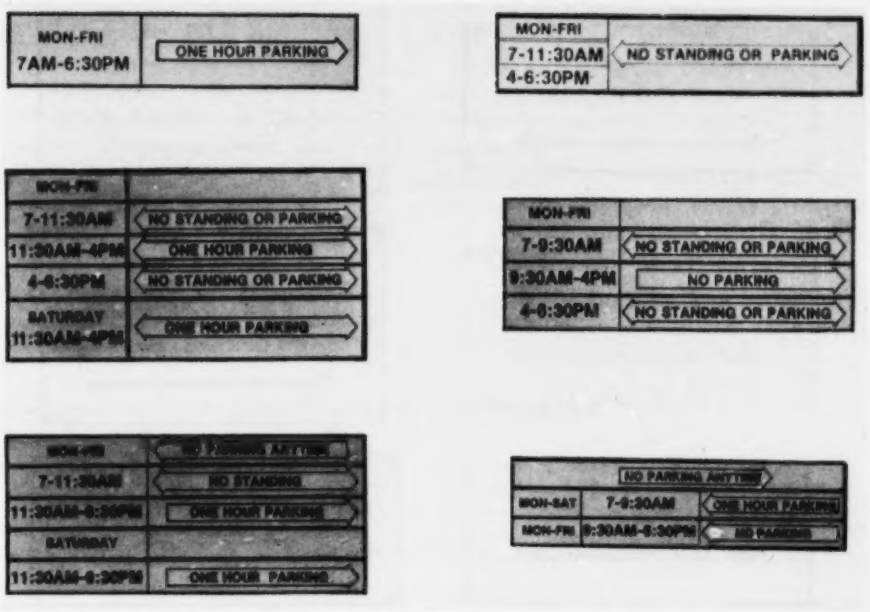


Figure 3.—Time-referenced series signs.

Table 2 also shows that subjects took longer to interpret time-referenced signs than conventional signs. However, the difference was not statistically significant.

Changeable message signs gave significantly fewer errors than the conventional signs and significantly more rapid responses than the other two series. Nevertheless, changeable message signs did give one error per 6.50 trials.

Errors sorted by single-, double-, and triple-paneled signs are shown in table 3. Triple-paneled signs showed almost as many errors as single- and double-paneled signs combined. On the average, subjects made an error every 2.5 trials on triple-paneled signs. It is not known if triple-paneled signs are difficult because of their layout or because they govern inherently complex situations. It is also not known why subjects responded most quickly to triple-paneled signs. It is recommended that triple-paneled signs be used as infrequently as possible.

Driver Difficulties With Parking Signs

Subjects' difficulties with the parking signs were further analyzed to determine the causes of the errors indicated in table 2. Five of the twelve questions and signs that gave the most errors concerned the parking-standing relationship. Of these five, four allowed standing even if parking was prohibited; the fifth prohibited both standing and parking. Two of the problem signs were conventional, two were changeable message, and one was time-referenced. Three signs (two conventional and one time-referenced) gave difficulty because Monday to Friday regulations conflicted with Saturday regulations.

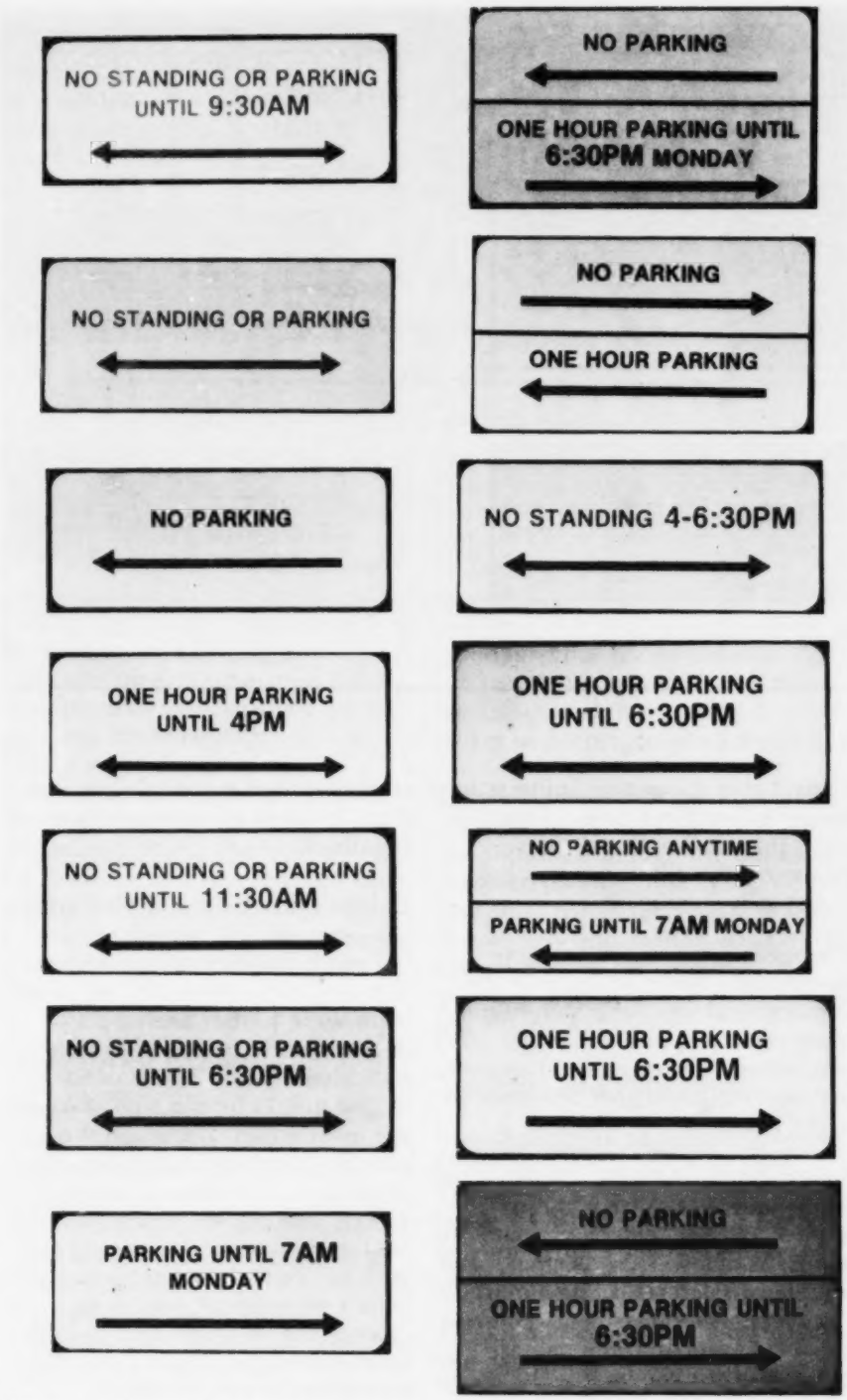

NO STANDING OR PARKING UNTIL 9:30AM
NO PARKING
ONE HOUR PARKING UNTIL 6:30PM MONDAY
NO STANDING OR PARKING
NO PARKING
ONE HOUR PARKING
NO PARKING
NO STANDING 4-6:30PM
ONE HOUR PARKING UNTIL 4PM
ONE HOUR PARKING UNTIL 6:30PM
NO STANDING OR PARKING UNTIL 11:30AM
NO PARKING ANYTIME
PARKING UNTIL 7AM MONDAY
NO STANDING OR PARKING UNTIL 6:30PM
ONE HOUR PARKING UNTIL 6:30PM
PARKING UNTIL 7AM MONDAY
NO PARKING
ONE HOUR PARKING UNTIL 6:30PM

Figure 4.—Changeable message series signs.

The other questions that gave difficulty concerned 2-hour parking and the rule that parking is allowed if not prohibited. One high error changeable message sign gave a premature warning of a regulation change, which was interpreted as having occurred.

Summary

Parking signs govern the stopping and standing of vehicles. Single-paneled parking signs described in the MUTCD are effective, but double- and triple-paneled signs, which present complex parking schedules, give difficulty. The concept was tested that drivers could better understand complex parking signs if the information was arranged by time because in interpreting a sign the driver must first determine the current regulations. Three time-referenced sign types that were translations of single-, double-, and triple-paneled conventional signs were experimentally compared. Measures of speed and accuracy were determined in a preliminary laboratory study. The most successful time-referenced sign was then tested against conventional parking signs and against changeable message signs. The changeable message signs showed only the regulations in force at the time of the question. These signs served as experimental controls to indicate how close time-referenced performance came to "best" performance. The following conclusions were made:

- Triple-paneled parking-standing signs give difficulty. Triple-paneled conventional signs give more errors than single- and double-paneled signs combined. Of the signs giving numerous wrong answers, five were triple-paneled signs, and one a double-paneled sign. No single-paneled sign had a high frequency of errors.

Table 2.—Parking sign study results

	Sign series		
	Conventional	Time-referenced	Changeable message
Total errors	231	174	133
Errors per question	0.27	0.20	0.15
Mean trials per error	3.74	4.96	6.50
Percent right corrected for chance	50.50	59.70	69.30
Mean response time (seconds)	6.03	6.67	4.28

Table 3.—Errors and response times related to number of sign panels

Sign series and number of panels	Total errors	Trials per error	Mean response time
			Seconds
Conventional			
1	55	5.2	6.44
2	62	4.6	6.51
3	114	2.5	5.13
Time-referenced			
1	46	6.3	7.01
2	48	6.0	7.73
3	80	3.6	5.28
Changeable message			
1	35	8.2	4.67
2	35	8.2	4.14
3	63	4.6	4.04

• Organizing parking sign information by time is favored. The average number of errors with time-referenced signs was three-fourths the number of errors with equivalent conventional signs. The findings of the study suggest that complex parking regulations might be better communicated by a rearrangement of the conventional MUTCD-approved signs to give prominence to time information. However, the adoption of time-referenced signs should not be expected to result in error-free interpretations.

• The parking-standing relationship should be clarified. Subjects' misunderstandings of these regulations were the most important source of error in the study. If standing is forbidden, the sign should read: "No Parking or Standing." The public also needs a clear definition of "standing."

• The signs should be simplified, and unusual signs should be avoided. Subjects had difficulty with 2-hour signs and with signs where Saturday regulations conflicted with Monday to Friday schedules. Although not specifically investigated in this study, it is recommended that "Entrance," "Loading Zone," "Embassy," and "Towaway Zone" messages should be used infrequently on parking signs. The signs are often already burdened with information.

These conclusions indicate public difficulty with many parking signs. However, most signs are of the easily understood, single-paneled type. Parked cars also serve as an important clue to parking permission, even if the sign itself is ambiguous. However, a regulatory parking sign must stand on its own. A driver who has attempted to read triple-paneled signs from a moving car will be aware of the difficulties investigated in the study.

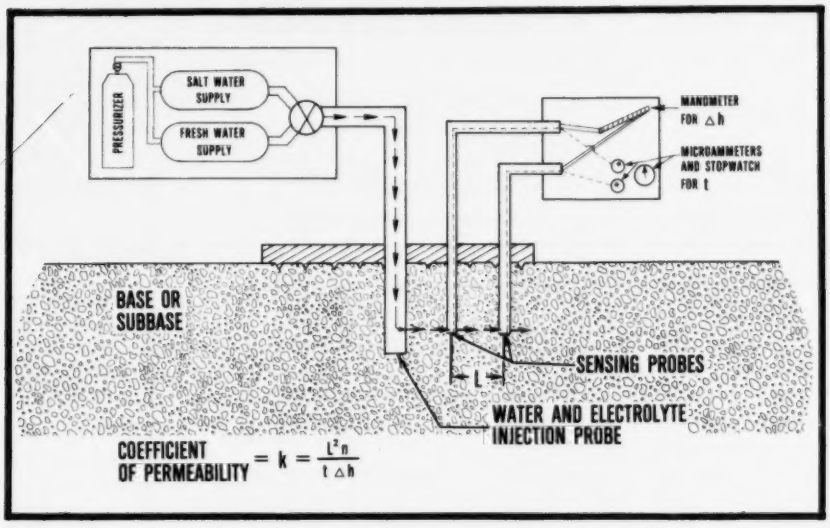
REFERENCES

(1) "Manual on Uniform Traffic Control Devices for Streets and Highways," *National Advisory Committee on Uniform Traffic Control Devices, U.S. Department of Transportation*, Washington, D.C., 1978.

(2) D. C. Hanson, C. Bennett, and G. Radelat, "Curb Parking Sign Study," Highway Research Record No. 151, *Highway Research Board*, Washington, D.C., 1966, pp. 18–40.

Donald A. Gordon is a research psychologist in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been involved in research on the driving process and the development of improved signs and roadway delineations. Dr. Gordon has published articles on topics such as perception in vehicular guidance, experimental isolation of the driver's visual input, and the contribution of psychology to the traffic flow theory. Lately, he has been concerned with the development of improved highway guide signs.

Determination of the In Situ Permeability of Bases and Subbases



by

Lyle K. Moulton and Roger K. Seals[1]

This article discusses the development and evaluation of the field permeability testing device (FPTD), a prototype in situ test device for determining the permeability of highway bases and subbases. The prototype device, based on the velocity method of in situ permeability determination, was evaluated extensively in the laboratory and field. Based on these evaluation results, it was concluded that the prototype FPTD satisfied stipulated performance criteria and provided a convenient, accurate, and reproducible means for determining the in situ coefficient of permeability of highway bases and subbases. However, the equipment and procedure need further development before the device can be adopted for routine use.

Introduction

Many of the problems associated with the unsatisfactory performance of pavement systems are attributable to excessive moisture in the subgrade and the structural section of the pavement. Recognizing this situation, many highway agencies specify that granular bases and subbases have outlets that drain water from the pavement section as quickly as possible. However, the base and subbase layers are not always permeable enough to prevent damage to the pavement under saturated conditions and normal traffic loads. (*1*) [2]

As part of the effort to develop effective subsurface drainage systems for highway pavements, rational procedures have been developed for estimating the amount of water that enters the pavement section and designing the drainage layers to rapidly remove this water. (*1*) The principal material property required to use these procedures is the coefficient of permeability. Minimum coefficients of permeability must be specified to meet design requirements of the drainage system.

[1] This article is a condensation of the report "Determination of the In Situ Permeability of Base and Subbase Courses," by L. K. Moulton and R. K. Seals, Report No. FHWA-RD-79-88, *Federal Highway Administration*, Washington, D.C., May 1979. The report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 300908).

[2] Italic numbers in parentheses identify references on page 143.

Existing laboratory testing methods can be used to determine the permeability coefficients of candidate drainage layer materials; these values can be compared with the design requirements. Although these laboratory methods are well known and quite reliable, relatively small variations in gradation and density, such as those normally encountered in construction, can produce large changes in the coefficient of permeability. (*2, 3, 4*) Thus, the field permeabilities of these materials can differ from the values determined in the laboratory. Therefore, the coefficients of permeability of base, subbase, and subgrade materials should be determined in situ. Proven methods for such determinations must be available before the coefficient of permeability can be routinely specified in construction contracts to control materials and construction practices.

Recognizing this need, the Federal Highway Administration (FHWA) contracted for the development of a test apparatus and procedure for determining the in situ permeability of highway base, subbase, and subgrade materials. In Phase I of the project, feasible measurement techniques were developed and investigated in the laboratory; the optimum technique was used as a basis for the Phase II construction, laboratory, and field evaluation of the prototype FPTD.

During the study, several important facts about the problem and the required operating characteristics of the prototype FPTD were given special consideration:

- The flow or seepage domain in pavement structural systems consists of layered, nonhomogeneous, anisotropic media. A knowledge of coefficient of permeability (saturated hydraulic conductivity) in the transverse or longitudinal direction (while keeping in mind that the vertical permeability influences the amount of surface water that infiltrates the pavement structural system) is of primary concern in designing the drainage of these layers. The overriding influence of the horizontal flow limits the usefulness of techniques that measure only vertical permeability.
- The prototype FPTD should measure the coefficients of permeability of individual layers ranging in thickness from 75 mm (3 in) to 460 mm (18 in), with the coefficients of permeability ranging from 10^{-4} cm/sec to 10 cm/sec (0.4×10^{-4} in/sec to 4 in/sec), under various boundary conditions. These boundary conditions included both initially saturated and unsaturated layers, as well as underlying layers that were either more or less permeable than the layer being evaluated.
- The measurement technique adopted should be nondestructive or disturb the layer being evaluated as little as possible.
- The prototype FPTD must be simple to operate and durable.
- The device should measure the coefficient of permeability within a factor of 2 of the true permeability 90 percent of the time.

Phase I of the research was completed in December 1977 and an interim report was published. (*5*) The final report for the project was published in May 1979.

The Velocity Technique

An intensive search of the research literature revealed no in situ permeability test method that met all the performance criteria necessary for highway bases and subbases. Several candidate methods were identified, however, and the project was directed toward the development and testing of a "velocity technique" for the in situ determination of the permeability of highway bases and subbases. (*6–11*)

Fundamental considerations

The velocity technique involves the direct application of Darcy's Law, which can be stated as follows:

$$v = ki$$

Where,

v = The discharge velocity.
k = The coefficient of permeability (in units of velocity).
i = The dimensionless hydraulic gradient.

The hydraulic gradient, i, can be expressed in finite difference form as follows:

$$i = \frac{\Delta h}{L}$$

Where,

Δh = The loss in total head between two points on a flow path (streamline) in the flow domain.
L = The distance measured along the flow path between these two points.

The discharge velocity, v, can be expressed as follows:

$$v = n \times v_p$$

Where,

v_p = The seepage velocity (the actual average velocity of flow within the pores of the soil).
n = The porosity of the soil.

Thus, Darcy's Law can be rewritten and solved for the permeability, k, to give the following:

$$k = \frac{n \times v_p}{\Delta h / L}$$

Further development of the velocity technique required the following: (1) A method for the establishment of saturated steady state flow within the layer of base or subbase to be evaluated; (2) a method for measuring the average seepage velocity, v_p, along a flow path between two points a known distance, L, apart; (3) a method for determining the head loss, Δh, between these two points; and (4) a method for determining the in situ porosity, n, of the base or subbase being evaluated.

Development of the technique

Studies were conducted to provide the equipment and procedures necessary for developing a practical in situ permeability measurement system based on the velocity technique. The following conclusions were reached:

- Saturated steady state flow could be established by injecting water under constant pressure through a perforated injection tube located in the center of a circular plate. Flow net studies, conducted using electric analogs, showed that the flow pattern and the configuration of streamlines and equipotential lines could be controlled by regulating the plate diameter and injection depth relative to the existing boundary conditions—that is, the base and subbase layer thicknesses and their relative permeabilities. Ideally, streamlines of the flow pattern within the layer being evaluated should be essentially parallel to the surface. Flow net studies showed that this condition was best achieved by injecting water over the full depth of the layer when an impervious or much less permeable layer is under it. Under these boundary conditions, full depth injection was not absolutely necessary to establish a sizable zone of essentially horizontal flow.

- Electrical conductivity could be used to measure seepage velocity. Once saturated steady state flow has been established, an electrolyte solution is introduced through the injection tube, and electric conductivity probes are used to time the flow rate between two selected points on a streamline. A two-wire probe and a microammeter system was simpler and more effective and economical for both laboratory and field use than were the other conductivity measurement systems investigated.

- Two possible methods were considered for determining the head loss, Δh, between the velocity measuring probes in the flow domain: (1) The analytical (mathematical) evaluation of the head loss between the two points as a function of the measured flow rate and total head loss through the system and (2) the direct measurement of the head loss between the two points. The analytical solution required knowledge of the ratio of the permeabilities of the various layers in the flow domain. Therefore, the head loss had to be determined by directly measuring the fluid pressures at the ends of the electric conductivity probes. Either a manometer or a differential pressure gage (DPG) could be used to measure the head loss.

- Using measured values of the dry density, γ_d, and the specific gravity of solids, G_s, of the soil, the porosity, n, could be calculated using the following expression:

$$n = 1 - \frac{\gamma_d}{G_s \times \gamma_w}$$

Figure 1 shows the principal components of the field permeability test device. As shown in figure 2, the pressure taps were incorporated into the electric conductivity probes.

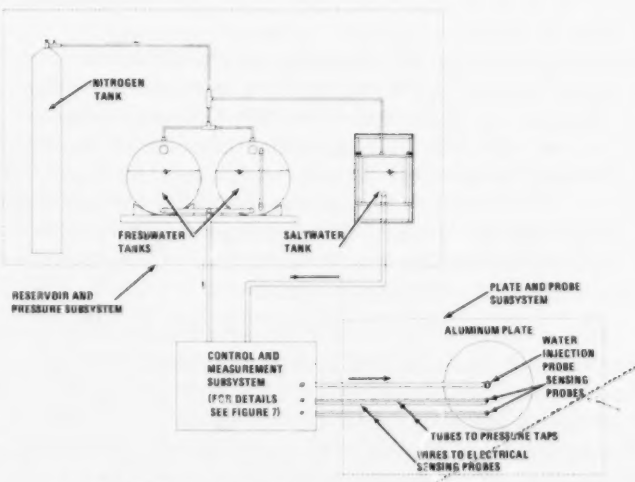
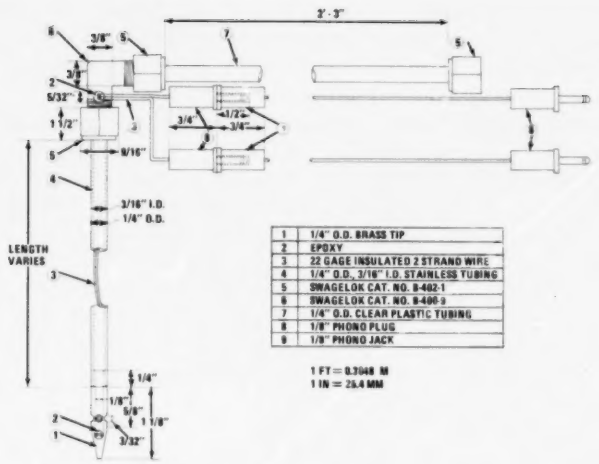


Figure 1.—The FPTD and its subsystems.



1	1/4" O.D. BRASS TIP
2	EPOXY
3	22 GAGE INSULATED 2 STRAND WIRE
4	1/4" O.D., 3/16" I.D. STAINLESS TUBING
5	SWAGELOK CAT. NO. B-402-1
6	SWAGELOK CAT. NO. B-400-9
7	1/4" O.D. CLEAR PLASTIC TUBING
8	1/8" PHONO PLUG
9	1/8" PHONO JACK

Figure 2.—Sensing probe.

Preliminary laboratory studies

The accuracy and reliability of the technique were studied initially by placing electric conductivity/pressure probes in the side of a modified constant head permeability test device and then comparing the coefficient of permeability values based on the velocity technique and the conventional constant head method. In addition to verifying the viability of the velocity technique, these studies produced the following two significant observations:

- Some error is introduced into the velocity determination by hydrodynamic dispersion and diffusion of the salt solution. Thus, the apparent velocity of flow between the two probes measured by the initiation of conductivity change was slightly higher than the actual average velocity determined in the constant head permeameter. The magnitude of this error was time dependent and could be minimized by using a combination of probe spacing and hydraulic gradient that allowed the test to be run in the shortest possible time.

- The best electrolyte solution tested consisted of 25 mg (8.75×10^{-4} oz) of ammonium chloride (NH_4Cl) in 100 ml (3 oz) of water.

The data, summarized in figure 3, compare the coefficient of permeability, k_{ch}, measured by the modified constant head device with that determined by the velocity technique, k_p, using the new electric probe with pressure tap system installed in the modified constant head permeameter. The linear regression equation (shown as the solid line in figure 3) can be expressed as the following:

$$k_{ch} = 0.64 k_p^{0.958}$$

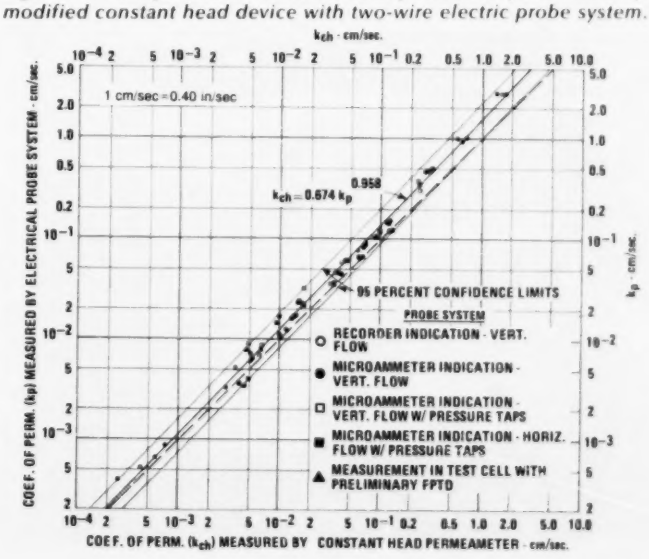
This equation fits the experimental data well, as evidenced by a coefficient of determination, r^2, of 0.991 and a coefficient of variation, CV, of 5.08 percent. The velocity technique slightly overpredicts the coefficient of permeability because of the dispersion and diffusion phenomena noted previously. Even with this problem, the data show that the two observations agree very well, which confirms the practicality of the velocity technique for in situ permeability measurements.

Before fully developing the prototype FPTD, a preliminary device was tested in 1.5 m × 1.5 m (5 ft × 5 ft) cells. (5) These cells were designed to allow direct determination of coefficients of permeability for simulated bases and subbases under saturated horizontal sheet flow conditions. Results obtained with the preliminary device agreed very well with the direct determinations when corrected to account for the effects of hydrodynamic dispersion and diffusion.

Development and Evaluation of the Prototype FPTD

The remainder of the project was devoted to completing the development of the prototype FPTD, evaluating the accuracy and reliability of the method, and establishing the suitability of the equipment for the field.

Figure 3.—Comparison of coefficients of permeability measured by modified constant head device with two-wire electric probe system.


k_{ch} - cm/sec.
1 cm/sec = 0.40 in/sec
$k_{ch} = 0.674\, k_p^{0.958}$
95 PERCENT CONFIDENCE LIMITS
PROBE SYSTEM
RECORDER INDICATION - VERT. FLOW
MICROAMMETER INDICATION - VERT. FLOW
MICROAMMETER INDICATION - VERT. FLOW W/ PRESSURE TAPS
MICROAMMETER INDICATION - HORIZ. FLOW W/ PRESSURE TAPS
MEASUREMENT IN TEST CELL WITH PRELIMINARY FPTD
COEF. OF PERM. (k_p) MEASURED BY ELECTRICAL PROBE SYSTEM - cm/sec.
k_p - cm/sec.
COEF. OF PERM. (k_{ch}) MEASURED BY CONSTANT HEAD PERMEAMETER - cm/sec.

Description of FPTD

The prototype device consists of three major subsystems: (1) The reservoir and pressure subsystem, (2) the control and measurement subsystem, and (3) the plate and probe subsystem (fig. 1).

The reservoir and pressure subsystem consists of the freshwater supply tanks, a saltwater supply tank, and a pressure source. The freshwater supply tanks provide the water for saturating the base or subbase and establishing steady state flow during permeability testing. The saltwater supply tank provides the electrolyte solution that initiates the conductivity change. The pressure source, which maintains the gas pressure of the system, is a regulated nitrogen gas cylinder. For field use, the reservoir and pressure subsystem was mounted in the rear of a West Virginia University (WVU) mobile research van.

The control and measurement subsystem consists of the hydraulic controls, the electric sensing system, and the pressure sensing system (fig. 4). The hydraulic controls can shut off the water supply or precisely regulate the flow of either freshwater or saltwater (electrolyte) to the water injection probe. The two electric sensing circuits provide an adjustable response to the conductivity change that occurs as the electrolyte solution passes the electrodes in the sensing probes. The pressure sensing system consists of a differential manometer that determines the head difference between the pressure taps in the sensing probes.

Figure 4.—Closeup of the control and measurement subsystem.

The plate and probe subsystem consists of the horizontal plate, water injection probe, and sensing probes. The 460 mm (18 in) diameter aluminum plate is equipped with a central port through which the water injection probe is inserted and with radially located ports through which the sensing probes are inserted. To minimize the amount of piping at the interface between the plate and the base or subbase, the plate was prepared with annular projections on its bottom surface.

The water injection probe consists of closed-end tubing with holes at regular intervals along its length. During testing, the probe is oriented so that these holes point in the direction of the row of plate ports, that is, in the direction of testing.

The sensing probes (fig. 2) were constructed from stainless steel tubing with a brass tip. The tip contains four ports to permit the access of water to the manometer for head measurement. A two-wire electrode is brought down through the sensing probe and secured in the tip. Both the water injection probes and the sensing probes were made in various lengths to suit the geometry of the test situation.

Operation of the FPTD

Once the plate has been positioned on the surface of the bases or subbases, the central water injection and sensing probes are inserted into predriven holes. The probes are positioned at preselected depths and oriented to produce a consistent flow/measurement direction. The fresh- and saltwater reservoir lines are connected through the control/measurement system to the water injection probe. The piezometer and electric conductivity lines are connected between the probes and the control/measurement system. The plate is then jacked against the test surface using the equipment vehicle for reaction. Figure 5 shows the test setup.

When freshwater begins to flow through the central probe, the development of saturated steady state flow is determined by observing seepage adjacent to the edge of the plate as well as by a stable differential manometer reading. The differential head, and thus the quantity of flow, can be adjusted by a regulator on the nitrogen pressure system. Once a stable flow is established, a "slug" of the salt solution is introduced into the injection probe flow system at the same head as the freshwater. The salt slug is followed by a continuous flow from the freshwater reservoir.

Timing is begun when the microammeter for the upstream probe deflects and is stopped when the microammeter for the downstream probe deflects. After

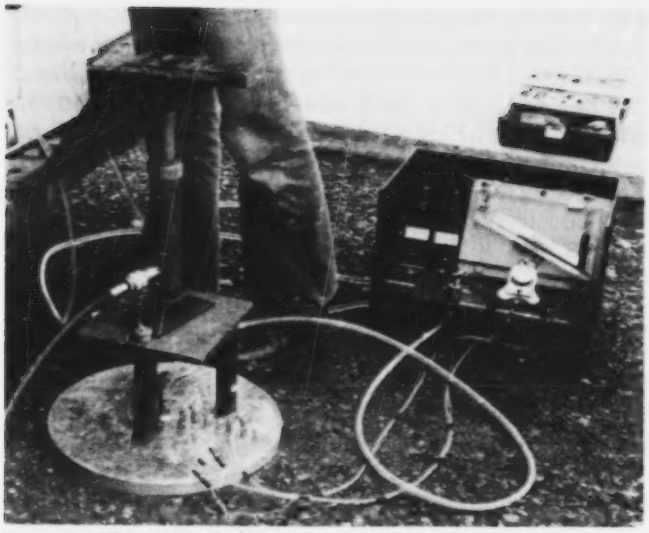


Figure 5.—Test setup before start of the flow of water.

the initial test, the freshwater that follows the salt slug "flushes" the salt solution from the flow domain causing the microammeter needles to return to their original undeflected positions. The data that are recorded and used for calculation of the coefficient of permeability are as follows: Travel time, t (seconds); differential head, Δh (centimetres); and travel distance, L (centimetres).

Once the system returns to the original condition, the flow (and thus the differential head) is adjusted and another test can be conducted.

Field evaluation of the FPTD

The performance of the FPTD was evaluated under actual field conditions. The field testing program consisted of investigations at 18 test sections in 13 locations in 8 States (Kentucky, Maryland, Michigan, North Carolina, Ohio, Pennsylvania, Tennessee, and West Virginia). Almost all of the test sections consisted of a single base or subbase layer overlying an impervious subgrade. Samples were collected at each test location and returned to the WVU laboratories for physical properties and permeability testing. In addition, field nuclear moisture-density determinations were made at each test location. These tests were conducted to calculate the in situ porosity of the layer and to provide the target dry density values for the laboratory permeability tests.

The FPTD performed satisfactorily in the field on all but 3 of the 18 test sections. In two tests, the flow required to maintain steady state flow in the base exceeded the flow capability of the FPTD. Laboratory time-lag permeameter tests (2) on these materials revealed average permeability coefficients of 2.76 and 8.27 cm/sec (1.1 and 3.3 in/sec), both near the proposed upper limit of the operating range for the equipment. The third failure occurred on a dense graded aggregate base that had a laboratory permeability coefficient of 2.73×10^{-3} cm/sec (1.1×10^{-3} in/sec), which is near the proposed lower operating limit for the equipment. In all three failures, excessive piping at the plate-base course interface was found to be one of the limiting conditions. As a result, the annual serrations previously noted were machined into the base of the plate in an attempt to develop a better interfacial seal. Although subsequent performance improved, none of the remaining test sections exhibited particularly high or low permeability values, so the interfacial seal was not tested under either extreme. It was also noted that a better interfacial seal could be achieved on freshly compacted material than on bases or subbases that had been in place and under traffic for some time.

Comparison of the field and laboratory results revealed that for 6 of the 14 measured coefficients of permeability (43 percent), the field values were within a factor of 2 of the laboratory values, and for 10 of the 14 measured coefficients of permeability (71 percent), the field values were within a factor of 10 of the laboratory values. These comparisons were not judged to be particularly good, but certain inherent problems exist in comparing field and laboratory test results. There may be differences in dry density (porosity), differences in direction of fluid flow, and differences in fabric (particle orientation). Even though the field results could not be directly correlated with the laboratory results on the same material, the laboratory evaluation (discussed below) on carefully controlled samples established that the prototype FPTD could produce consistent and accurate results over a relatively wide range of material types and boundary conditions.

The field evaluation program did identify potential minor problems with the equipment, demonstrate its ruggedness and durability, and assist in developing a systematic test procedure. The program also identified possible refinements to improve equipment portability and operation. The sensing probes were the only components that presented problems during the field tests, but the probes are inexpensive and easy to replace when failures occur.

Although the subbase layer in Tennessee contained 5 percent calcium chloride for stabilization, there was no problem in achieving steady state conditions and obtaining a satisfactory permeability measurement.

Combined setup and testing times for a given layer ranged from 45 to 90 minutes.

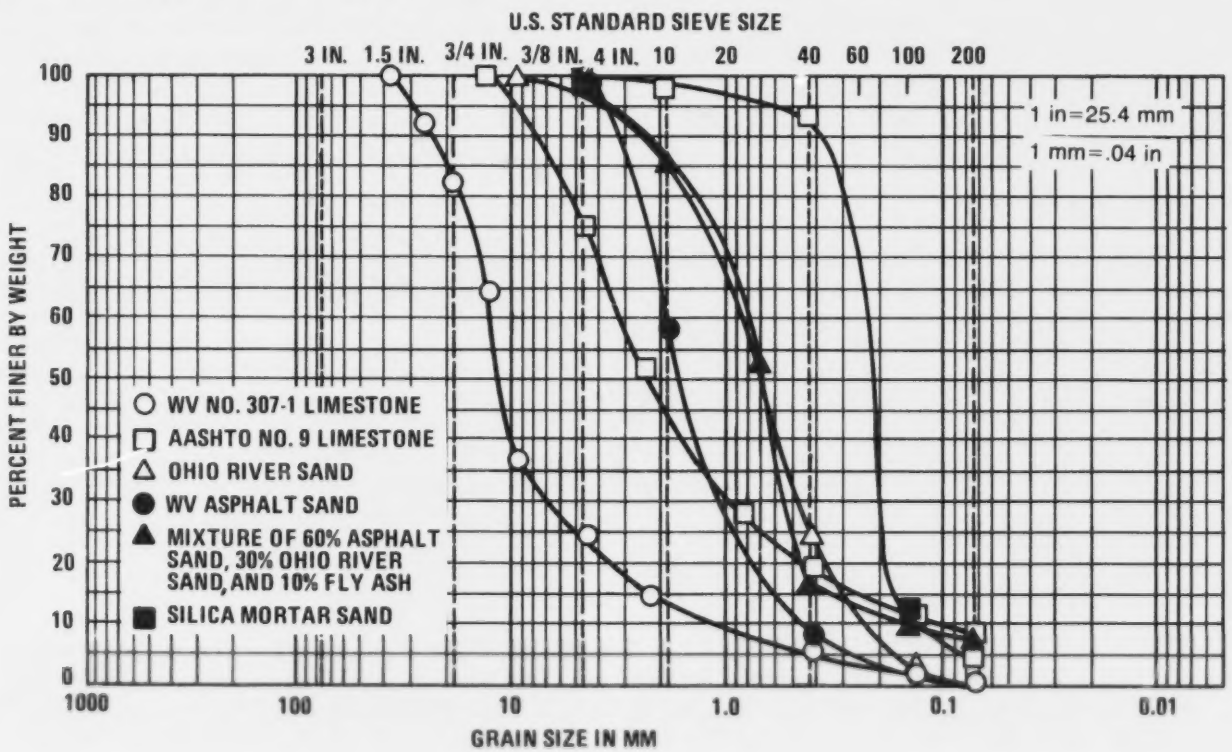
Laboratory evaluation of the FPTD

Because of the difficulties anticipated in trying to correlate field and laboratory results, the study included a laboratory evaluation of the FPTD. The prototype FPTD's performance was evaluated for a variety of materials and physical and hydraulic conditions.

Six different materials were selected, more on their permeability—between 10^{-4} and 10 cm/sec (0.4×10^{-4} and 4 in/sec)—than on their suitability as a base or subbase. These materials were as follows, in order of decreasing permeability: (1) A crushed limestone aggregate, satisfying West Virginia Department of Highways (WVDOH) 307–1 base course requirements, (2) a crushed limestone aggregate meeting the American Association of State Highway and Transportation Officials (AASHTO) No. 9 gradation requirements (washed), (3) a limestone sand meeting the WVDOH requirements for fine aggregate in asphaltic concrete, (4) an Ohio River sand meeting the WVDOH requirements for fine aggregate in portland cement concrete, (5) a silica mortar sand supplied by the WVU physical plant, and (6) a blend of 60 percent asphalt sand, 30 percent Ohio River sand, and 10 percent fly ash. The average test-cell-determined coefficients of permeability in cm/sec of these materials were, respectively: (1) 3.56 (1.4 in/sec), (2) 2.70×10^{-1} (1.1×10^{-1} in/sec), (3) 1.10×10^{-1} (0.43×10^{-1} in/sec), (4) 3.51 to 5.90×10^{-2} (1.4 to 2.3×10^{-2} in/sec), (5) 1.80×10^{-2} (0.7×10^{-2} in/sec), and (6) 3.75×10^{-3} (1.5×10^{-3} in/sec). The grain size distribution curves for these materials are shown in figure 6.

Physically, the boundary conditions were varied by using either a homogeneous or layered system with different layer thicknesses. The physical location of impermeable, moderately permeable, and very permeable materials was varied within the layered system. The location of the free water surface was varied relative to the position of the layer being tested. In addition, the effects of FPTD test parameters, such as direction of testing, plate size, probe location, and probe spacing, on permeability coefficients were evaluated in the laboratory.

Figure 6.—Grain size distribution curves for the materials used in the test cells to evaluate the FPTD.



The test results indicated that neither layer thickness nor water table depth significantly affects the measured coefficient of permeability if saturated steady state flow can be established. Similarly, if the underlying material was either impervious or moderately permeable, it did not significantly influence results.

Some problems were encountered in obtaining reliable measurements in 75 mm (3 in) and 150 mm (6 in) layers of Ohio River sand overlying a 300 mm (12 in) layer of WVDOH No. 307-1 crushed limestone. In this test, the permeability of the underlying layer was over 100 times that of the layer being tested. Consequently, the bulk of the flow was found to move abruptly downward and then horizontally through the very pervious layer. Other tests showed, however, that permeability measurements of a layer that is above a much more permeable layer are reliable if a condition of steady state saturated flow is maintained in both layers.

The influence of the direction of testing was evaluated by running tests at the same location and placing the sensing probes at different directions. As expected, some materials exhibited considerable variations, whereas others showed only small differences. These results suggest that considerable anisotropy might exist, particularly under field conditions where placement and compaction of bases and subbases might not be as uniform as in the laboratory test cells. Consequently, in actual practice it would be desirable to perform tests in orthogonal directions at a given test location and then take the permeability as the geometric mean of the results.

The influence of the plate size, the depth of the water injection probe, and the depth, location, and spacing of the sensing probes are interrelated. Ideally, these parameters should be controlled to produce a zone of essentially horizontal flow so that the tips of the sensing probes can be located on a single streamline. Plate size can be held constant if the depth of the water injection probe and the depth and location of the sensing probes are properly controlled. Therefore, the 460 mm (18 in) diameter plate was selected for most of the laboratory and field tests.

For layers 300 mm (12 in) thick or less, full depth penetration of the water injection probe was required. For layers from 300 to 460 mm (12 to 18 in) thick, a 300 mm (12 in) penetration depth was found to be satisfactory.

For layer thicknesses less than 300 mm (12 in), placing the sensing probes at the center of the layer proved satisfactory. For layer thicknesses exceeding 300 mm (12 in), the sensing probe depth should not exceed 150 mm (6 in). However, averaging measurements made at several depths is desirable for such layers.

Results of tests to determine the influence of sensing probe location and spacing were somewhat erratic but showed that better results were produced with 50 mm (2 in) and 75 mm (3 in) probe spacing, with the interior probe within the center third of the plate, that is within 75 mm (3 in) of the center. This was particularly true where a layer of highly permeable material is under the layer being tested.

The overall accuracy of the FPTD and the reproducibility of the results achieved were evaluated by comparing coefficients of permeability measured by FPTD, k_d, with the coefficients of permeability for the same materials measured in the test cells, k_{tc}, using steady sheet flow. These data are compared in figure 7. The linear regression equation (shown as the solid line in figure 7) can be expressed as follows:

$$k_d = 1.023\, k_{tc}^{1.023}$$

The equation fits the experimental data well, as evidenced by a coefficient of determination, r^2, of 0.959. On the average, the FPTD produced values very close to those measured by the test cell technique, with only a slight tendency for underprediction in the low permeability range and a very slight tendency for overprediction in the high range. The overprediction of coefficient of permeability, which showed up earlier as a result of dispersion and diffusion phenomena, did not show up here.

Figure 7 was prepared and the statistical analysis was performed using all 410 test results that were obtained during laboratory evaluation of the FPTD. Although the accuracy of certain data was questioned, the data were included in figure 7 and the analysis so that the resulting comparison between k_d and k_{tc} would represent the "worst condition" and thus could represent the lower limit of the accuracy and reproducibility of the values obtained with the FPTD.

The shaded area in figure 7 represents the zone where the data would fall if the FPTD measured the coefficient of permeability within a factor of 2 of the true value. When all of the test results are considered, 336 of the 410 test values (82 percent) fall within the shaded zone. However, as noted earlier, some of the test data can be questioned, including data obtained with what proved to be unsatisfactory locations and spacings of the sensing probes. If these 44 pieces of data are excluded from the total, then 336 of the remaining 366 test values (91.8

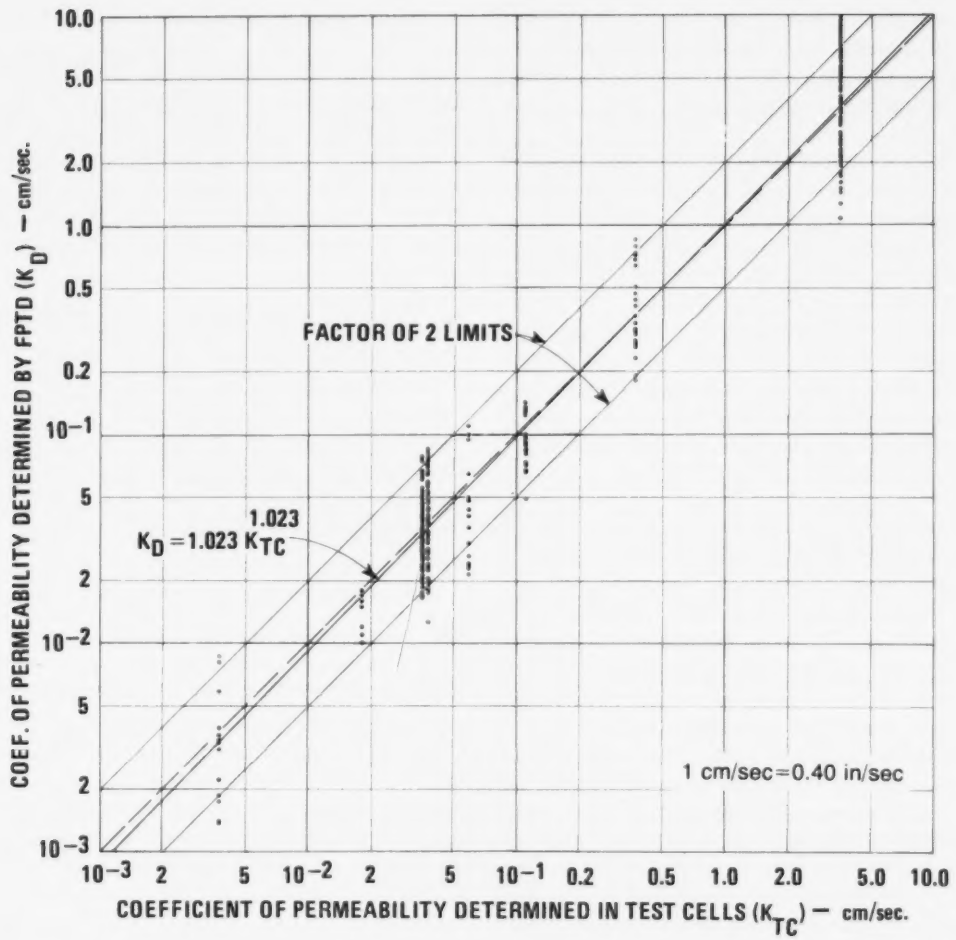


Figure 7.—Comparison of permeability determined in test cells with permeability measured by FPTD.

percent) fall within the shaded zone. Thus, the prototype FPTD satisfies the requirement that it measure the coefficient of permeability within a factor of 2 of the true value 90 percent of the time.

Conclusions and Recommendations

The prototype FPTD can measure, with satisfactory accuracy and reproducibility, the coefficient of permeability of typical bases and subbases under a variety of physical and hydraulic boundary conditions. Measurements are reliable if a zone of saturated steady state flow can be established and maintained during testing.

Although the FPTD can measure the coefficient of permeability of bases and subbases within a practical range, at the present state of development it cannot measure permeabilities below about 10^{-4} cm/sec (0.4×10^{-4} in/sec) or above about 10 cm/sec (4 in/sec).

The equipment proved to be rugged, reliable, and easy to operate. In addition, the components are inexpensive and easy to construct.

Although the results of the investigation were generally favorable, additional equipment and procedural improvements are needed before the test is routinely used. The FPTD should be widely field tested under

diverse conditions that might be encountered in routine use. FHWA is currently initiating an evaluation program in which the prototype FPTD is being loaned to State highway agencies for field testing.

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Lyle K. Moulton is a professor of civil engineering at West Virginia University where he specializes in geotechnical engineering. His principal areas of interest are groundwater and seepage, earthwork design, and foundation engineering. His recent research work includes studies on in situ permeability measurement and tolerable bridge movements.

Roger K. Seals is a professor of civil engineering at West Virginia University where he specializes in geotechnical engineering. His principal areas of interest are soil properties and behavior and foundation design. Much of his recent research has been devoted to the use of coal-associated residuals as construction materials.

The Seventh Annual FCP Conference

by

Debbie DeBoer

Although the near completion of the Interstate Highway System has led to the general assumption that an intensive program of highway research and development (R&D) is no longer justified, the opposite is true—highway R&D is needed more urgently now than ever. The highway program must redirect itself toward goals that reflect the changing complexion of the transportation field: Use of smaller cars; the fuel crisis; a greater number of pedestrians, bicyclists, and moped users; the need for rehabilitation of the existing highway system; and the ever increasing interest in and importance of safety for users of all modes of transportation. (*1*)[1]

As a result, R&D is still an important function of many highway agencies. Because this is the case, the efforts of various highway agencies and those of the Federal Highway Administration must be coordinated to eliminate duplication and insure proper direction of R&D efforts. This coordination is the mission of the Federally Coordinated Program (FCP) of Highway Research and Development[2] of the Federal Highway Administration.

Because participants in the FCP include R&D personnel from diverse areas of the highway community, communication among participants is vitally important. For this reason, annual review conferences have been held since 1973. The objective of the FCP conference, in review of selected projects from the FCP, is threefold: (1) To bring together members of each project team in an atmosphere of open discussion; (2) to test the research approach of each project by discussion with individuals from State highway agencies, operating offices, and FHWA field and headquarters offices; and (3) to test the plans for appropriate implementation of results.

Each project reviewed at the FCP conference goes through one of the three types of reviews: Initial, midstream, or final. An initial review is conducted when a new project has been outlined by FHWA staff and a budget request has been placed for the project, but before a significant level of contract funds is available. When approximately half of the planned contract funds have been committed, substantial Highway Planning and Research Program (HP&R) activity has been initiated, and substantive intermediate results have been obtained for a project, a midstream review is conducted. This review is held no later than the third year of funding. A final review is held when essentially all contract research has been completed, major implementation efforts are in preparation or in process, and HP&R activity on the project has passed its peak. The final review must be held no later than the second year after final contract funding.

The seventh annual FCP Review Conference was held in Williamsburg, Va., the week of December 3, 1979. Eighteen projects were reviewed in 15 sessions. The first half of the week dealt mostly with the review of projects from FCP Categories 1 and 2 ("Improved Highway Design and Operation for Safety" and "Reduction of Traffic Congestion, and Improved Operational Efficiency"). One project from Category 5 ("Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety") also underwent review during the first half of the week. Except for review of one project in Category 2, the second half of the week concentrated on Categories 3 ("Environmental Considerations in Highway Design, Location, Construction, and Operation"), 4 ("Improved Materials Utilization and Durability"), 5, and 7 ("Improved Technology for Highway Maintenance").

The review sessions began with an introduction and project overview by the project manager. These were

[1] Italic numbers in parentheses identify references on page 148.

[2] For further information on the FCP, write to the Associate Administrator for Research and Development (HRD-3), Federal Highway Administration, Washington, D.C. 20590.

followed by presentations on different elements of the project. After these presentations and some general discussion, the reviews of most of the projects followed a format different from that of previous FCP conferences. The attendees of the reviews met in small groups (breakout sessions) to discuss, critique, and make recommendations on the projects. Attendees then reconvened to review the discussions conducted in the breakout sessions. In past years most review sessions consisted of only large group discussions, and this year's change in format was to promote more open, indepth discussion of the projects under review. The general reaction of the conference attendees was that the review sessions benefited from this format change.

Table 1 lists the projects reviewed at the 1979 conference, their titles, and the type of review conducted for each project. Following is a brief description of the topics discussed at each project review, in some cases including problems that were encountered and recommendations that were made.

Table 1.—Projects reviewed at the 1979 FCP Review Conference

Project	Title	Type of review
1A	Traffic Engineering Improvements for Safety	Midstream
1J	Improved Geometric Design	Final
1N	Safety Improvements for Bicyclists, Moped Operators, and Pedestrians	Initial
1T	Advanced Vehicle Protection Systems	Midstream
1W	Measurement and Evaluation of Pavement Surface Characteristics	Midstream
2C	Requirements for Alternate Routing to Distribute Traffic Between and Around Cities	Midstream
2D	Priority Techniques for High Occupancy Vehicles	Final
2N	Improved Traffic Signing and Motorist Information Systems	Initial
3H	Social and Economic Considerations in Highway Development and Improvement	Initial
4C	Use of Waste as Material for Highways	Midstream
4K	Cost-Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments	Initial
5E	Premium Pavements for Zero Maintenance	Midstream
5K	New Bridge Design Concepts	Midstream
5L	Safe Life Design for Bridges	Midstream
7A	Physical Maintenance	Midstream
7B	Traffic Services	Midstream
7C	Management	Midstream
7D	Equipment	Midstream

FHWA project manager Ken Clear leading review session of Project 4K.

- 1A— In this review, the attendees discussed the project's 15 ongoing research studies, the process by which such studies were selected and evaluated, and how the results of such studies are implemented. Changes will be made to the direction and scope of five of the studies, as well as to the solicitation and evaluation process for candidate 1A studies. It was felt that the solicitation notice did not reach enough local jurisdictions and an effort should be made to correct this situation.

- 1J—Discussions in this session exhibited a concern for improved design and operation of horizontal curves especially in relation to superelevation, spirals, length of

curves, and transitions. Also, an evaluation of the design elements proposed in the new American Association of State Highway and Transportation Officials' (AASHTO) design guide was recommended.

• 1N—One problem which arose in this review was that of transmitting research results to the local level. The attendees were also concerned with improving followup efforts to determine how research is being implemented and what further research is needed in Project 1N. It was recommended that the Federal Highway Administration work jointly with the National Highway Traffic Safety Administration on future R&D efforts within this project.

• 1T—Computer simulation programs were discussed in this review and the consensus was "let the user beware." The development of these programs is in its early stage and many pitfalls may be encountered due to the current form and state of validation of the programs. Accurate input parameters must be determined for these programs and computer results must be compared carefully with experimental test data to assure reasonableness of results. Attendees did support further research with these tools because of the programs' potential for solving many highway safety problems.

• 1W—Skid resistance and hydroplaning, pavement texture, and road roughness were discussed in the breakout sessions of this project's review. There was also considerable discussion on the need for determining the hydroplaning potential of pavements by texture or speed gradient measurements. The policy implications of making the latter measurement with a bald tire to obtain greater sensitivity caused a lively interchange of ideas and opinions.

Review session of Project 5L being led by Jerar Nishanian, FHWA project manager.

• 2C—This review included intense discussions on motorist information systems and the possibility of overcoming problems that arise as an attempt is made to lessen congestion on highways. These problems include resistance by residents near freeways to improved lighting, perceived inequities between suburban and downtown ramp metering users, diversion from congested toll facilities to nearby free highways, and the need for commitments from politicians and transportation officials for funds for operation and maintenance of new traffic management and information systems.

• 2D—One recommendation arising from this review was that further research is needed on problems associated with operating high occupancy vehicle (HOV) facilities, such as accident prediction, the need for improved equipment for inserting and picking up HOV lane stanchions, and what future actions can be taken when HOV capacities are reached. Personnel from field offices indicated that the "users guide" which is being developed on priority techniques for HOV's will be most welcome if it presents recommendations that the user can accept, reject, and/or modify.

• 2N—In this review, it was recommended that more research be done to understand how drivers find unfamiliar destinations because drivers appear to use various methods when trying to find an unfamiliar location. A knowledge and understanding of these methods is necessary to the design of navigational aids so the new aids will be accepted and used by the motoring public.

• 3H—Discussions in the breakout sessions of this review resulted in recommendation of the following activities: Identify examples of

Monday morning session break.

successful social and economic (SE) analyses in the environmental impact statement; identify examples of effective early community involvement at the systems level; perform detailed research on survey techniques, existing and proposed, for SE analyses; perform research on the effectiveness of citizen involvement; and determine the value of predictive models for SE impact identification.

• 4C—It was concluded at this review that further study of waste materials such as fly ash and incinerator residue should be conducted by the States by experimental application. However, there is still a need to continue to develop mix design methods using reclaimed asphaltic concrete so a mixture having the physical and chemical properties and durability of virgin asphaltic concrete mixes can be produced.

• 4K—The need to preserve the national investment in highway bridges by stopping the corrosion of reinforcing steel in decks and other structural members caused considerable discussion at this review. The attendees agreed on the importance of further development of simple, lightweight cathodic protection systems to save structures already contaminated by road salt, but on which the process of spalling and delamination has not yet occurred to any major degree.

• 5E—In general, the attendees of this review were appreciative of the results that have already been achieved by this project, especially those showing that high traffic density pavements may warrant greater initial investment to reduce "life cycle" costs. Strong interest was shown in the adaptation of zero maintenance pavement concepts to the pavement rehabilitation process; this would result in an extension of the benefits of this project.

Project 1J review being led by George Pilkington II, FHWA project manager.

• 5K—The recommendations resulting from this project review included the consensus that the project is necessary and important, that it should be expanded to be truly representative of new bridge design concepts in both concrete and steel, and that inspection problems on existing concrete bridges should be transferred to Project 5L.

• 5L—During this review, a great deal of attention was focused on a fracture control plan for new steel bridges with fracture critical members. Most of the participants agreed that more work is needed both on determining the significance of the fracture toughness characteristics of the bridge steels used and on the level of quality control and inspection required during the fabrication of steel bridge members. There was also considerable discussion on how to avoid critical details created by improper design which no amount of quality control or toughness in the steel can make completely safe.

Breakout session of Project 1A.

Project 1N review being led by FHWA project manager John Fegan.

• 7A, B, C, D—The following subjects under the present maintenance R&D program were discussed: Equipment Management System—Test and Evaluation; Litter Pickup Machine; Pavement Patching Demonstration and Evaluation; Salt Brine Deicer; Value Engineering of Selected Maintenance Activities (Rest Area Maintenance); and Mechanization of Pothole Patching. In addition to these reviews of Category 7 projects, the steering committee for the Highway Maintenance Research Needs Study met to begin work on developing a new list of R&D maintenance needs that will replace the list of 27 items already acted on. State and field maintenance personnel will be involved as much as possible in developing the new list to insure immediate, effective response by R&D to future maintenance needs. (2) The general framework for regional needs workshops was planned and meeting locations were established, including Atlanta, Ga.; Hartford, Conn.; San Francisco, Calif.; and Champaign, Ill. A national meeting is scheduled for August 1980 in Minneapolis, Minn., at which the maintenance R&D needs for the next 5 to 10 years will be finalized.

In addition to the discussions in these formal review sessions, there was opportunity for informal discussion during the session breaks and the two receptions held during the week. As in past years, the 1979 FCP Review Conference was a forum for discussion and exchange of information between research and operations personnel and gave participants a chance to evaluate and discuss the progress made in the FCP since its formation in 1970.

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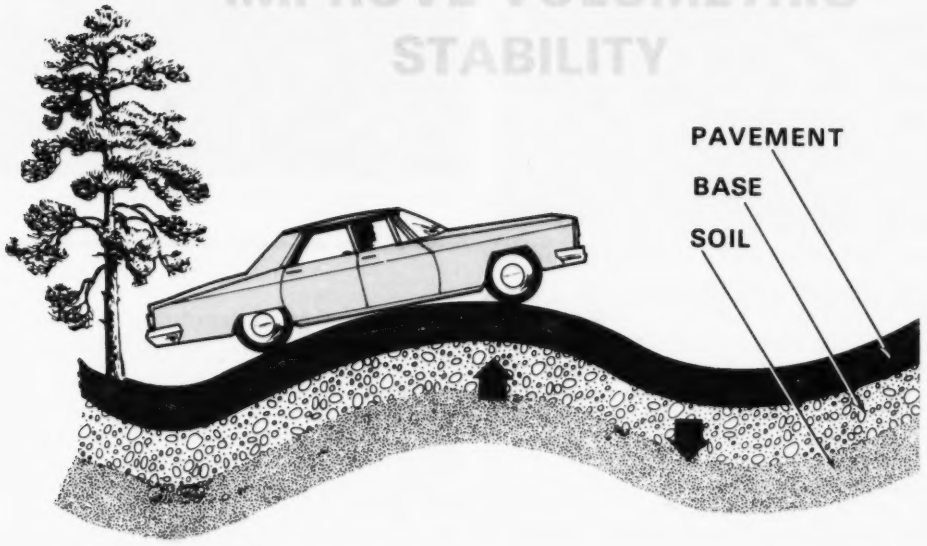
Debbie DeBoer is a technical publications writer-editor in the Engineering Services Division, Office of Development, Federal Highway Administration. She has been with the FHWA since 1975 and is editor of this journal.

The Frost Action Problem—An Overview of Research to Provide Solutions



by
Albert F. DiMillio and Donald G. Fohs

Introduction

The ravages of frost action on roads and streets is well documented by the news media each spring and by county and district maintenance engineers throughout the Northern States. For example, The Road Information Program (TRIP) estimated that $1.8 billion was required to rebuild the 9 600 km (6,000 miles) of pavement in the Northern United States that was destroyed during the winter of 1977. This expenditure was in addition to the cost of filling potholes and surfacing pavements with minor damage. (*1*)[1] According to the TRIP report, automobile repairs resulting from rough pavements and increased cost to transport goods because of detouring to avoid damaged roads should also be added to the $1.8 billion to full assess the impact of severe winter weather.

The problem of maintaining roadways and airfield pavements in areas of seasonal frost has long been a major concern to pavement design engineers. Although pavement designers have attempted to provide protection against the detrimental effects of frost action, the severe winters of the past several years have demonstrated that a basic and rational methodology for analyzing and designing pavement systems in cold regions does not presently exist.

The detrimental effects of frost action on subgrade soils contribute significantly to pavement distress and failure. During the past 5 years, Federal and State transportation agencies have made an effort to mitigate the effects of frost action on subgrade support capacity. Although the solutions have not been verified, it is important to periodically review current research efforts to assess the progress that has been made and to examine future plans that might need to be altered or redirected.

Proper coordination of the overall research and development (R&D) efforts is essential for efficient and prudent use of government funds and resources to solve the frost action problem. Various States and the Federal Highway Administration (FHWA) have joined forces

[1] Italic numbers in parentheses identify references on page 158.

through the Federally Coordinated Program (FCP) of Highway Research and Development to develop a comprehensive and well-coordinated program of R&D in this area. Also, in recent years the Federal Aviation Administration (FAA) and the U.S. Army Corps of Engineers (OCE) have joined forces with the States and FHWA to address this complicated problem. Current cooperative efforts of these Federal agencies extend previously separate R&D programs in this area. Significant past contributions of State highway department and university researchers also have been incorporated into the current program, and efforts are underway to expand the interest and involvement of these researchers.

Before 1972, FHWA played a passive role in the development of new technology and improved predictive techniques to address frost action. The principal contribution of FHWA's Office of Research was to coordinate the efforts of the various State highway agencies and universities conducting separate research studies in this area. As the number of studies increased and significant interest was generated during the Interstate construction program, FHWA researchers became actively involved in the overall effort.

In August 1973, FHWA organized a frost action research conference in Kittery, Maine, to discuss the status of ongoing research. An important objective of this conference was to determine the interest of the highway research community in participating in a large field investigation program for correlating actual heave measurements with the various available predictive techniques.

The Kittery conference brought together practicing engineers and researchers from universities, State highway agencies, FHWA, FAA, and OCE. It was agreed that field data on actual measurements of frost heave and the resulting loss of bearing capacity (thaw weakening) were needed to compare with predicted values. The consensus was that sections of inservice pavements be selected as sites for sampling, testing, and periodic measurement of frost heave. FHWA's Office of Research was assigned the task of planning an experiment to acquire and correlate the data.

FHWA initiated a Federally coordinated effort to develop a field testing program to validate or disprove existing prediction methods and stimulate the development of improved techniques. This article discusses the development of the field test program and the new predictive techniques.

Figure 1.—Typical pothole problem.

Figure 2.—Typical edge cracking problem.

In 1977, FHWA organized a frost action research review meeting at the FCP conference[2] in Atlanta, Ga., to review the progress of current efforts and plan for future investigations. Plans for many of the current research efforts were formulated or conceptualized at this meeting.

Nature of the Problem

The seasonal variation in the serviceability of a pavement is very pronounced in areas subject to alternating

[2] FHWA conducts annual research review meetings to assess the progress of the Federally Coordinated Program of Highway Research and Development.

Figure 3.—Concrete pavement breakup.

Figure 4.—Differential surface frost heave.

freezing and thawing. The combination of freezing and thawing of pavement subgrades is commonly called frost action. Differential pavement surface heaving (poor rideability) frequently is the effect of freezing, and subsequent thawing may lead to a greatly reduced load-carrying capacity—thaw weakening. (2) Many potholes and other pavement breakups (distress) result from thaw weakening (figs. 1–3).

Some soils are more susceptible to frost action than others and the amount of heave that occurs is not a good indication of how much strength loss will occur during the thaw period. It is also probable that a soil of lower frost susceptibility will experience greater heave than a soil of higher frost susceptibility if placed under more adverse temperature and water conditions. In fact, a highly frost-susceptible soil will not heave at all if either one of these two conditions (temperature and water) is missing. Conversely, clean granular materials not normally classified as frost susceptible will heave if the temperature and water conditions are sufficiently adverse.

Differential heaving

Because the amount of heave is dependent on three conditions that can be quite variable—frost susceptibility of the soil, freezing temperatures, and access to groundwater—uniform heaving cannot be expected. The differential heaving that results causes surface irregularities and general surface roughness in the form of bumps, waves, and distinctive cracking (fig. 4). Severe cases of differential heave will usually reduce traffic speeds significantly and may cause damage to vehicles or loss of control of the vehicle. The potential for abrupt differential heave at cut-to-fill transitions or culverts requires special design considerations. (2)

The amount of heave that occurs is not entirely a result of the expansion of free water in the soil voids when freezing temperatures penetrate the subgrade soil mass. This can often be a small percentage of the total heave. In severe cases of heaving, the extent and rate of growth of ice lenses are determined by the soil's ability to draw water from below by capillarity and also by the rate and depth of penetration of the freezing temperatures. The formation of ice lenses responsible for heaving is governed by the interaction of heat and mass transfer (moisture movement) in porous soil media—a very complicated phenomenon.[3] The growth of ice lenses also results in decreased soil density. After several cycles of freezing and thawing, the soil fabric can be adversely changed depending on the type of soil and the amount of ice lens buildup.

Thaw weakening

Thaw weakening is considered by many to be the more critical manifestation of frost action. It is just as complicated as the heave problem and it is probably more difficult to evaluate or predict. This problem occurs when the ice lenses formed in the subgrade during freezing begin to thaw from the surface downward. This thawing results in melt water being trapped between the pavement and the still frozen portion of the subgrade, which accompanied by loading of the soil in its loosened state generates excess pore water pressure and a corresponding decrease in load-carrying capacity. The duration and frequency of load application (static versus

[3] "Mathematical Modeling of Simultaneous Transport of Moisture and Heat in Soils," by B. J. Dempsey. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

dynamic) may also impact load-carrying capacity; that is, dynamic loading may be more destructive to soil structure, thereby reducing strength.

The increase in water content resulting from freezing and subsequent thawing is more detrimental for some soils than for others. The stability of fine-grained soils is more sensitive to changes in moisture content than is the stability of granular materials; that is, very slight increases in moisture content for silts and clays significantly decrease their stability while similar changes in water content do not affect the performance of granular materials. However, soil moisture content cannot be used as an indirect measure of thaw weakening because certain clay soils lose significant supporting capacity during thawing without significant increase in bulk moisture content. (*2*) A different set of physical and environmental factors influence heave and cause subgrade weakening, thus requiring separate evaluations.[4]

Frost susceptibility indicators

The frost action problem is comprised of a series of interdependent factors or parameters that vary over a range of values. Understanding the mechanism of frost action in soils requires a knowledge of soil behavior including soil physics. The frost susceptibility of soils is still a relatively unknown quantity; however, it is generally recognized that a soil is susceptible to frost action only if it contains fine particles. Most studies have shown that soils free of fines (particles smaller than the 200-mesh sieve) do not develop significant ice lens buildup or ice segregation. As a result, the engineering community has used various indirect measures or indicators based on particle-size distribution, pore-size distribution, grain shape, and plasticity characteristics. All of these contribute to frost susceptibility or ice lens buildup in varying degrees. One of the best known indicators is the Casagrande Criterion. (*2*)

The Casagrande Criterion, based on particle-size distribution, gives a high assurance that frost action problems will not occur; however, in many cases it rejects soils that are not a problem in the frost environment. Because sieve analyses are simple and inexpensive to run, many highway engineers prefer to use the Casagrande Criterion when an unexpected change in the soil profile is uncovered during construction. An answer is soon given on the soil's suitability for use within the roadway.

To avoid detrimental effects of frost action, most agencies require that materials within the frost penetration zone be nonfrost susceptible. This frequently requires that subgrade soils be excavated and replaced with frost-free material. Because excavation and replacement with frost-free material is expensive, and the material being replaced may not be an adequate performer, researchers are developing more accurate and direct methods of measurement. Most of these methods involve the unidirectional freezing of a soil sample and the measurement of heave rate, heave pressure, and moisture tension.

CRREL test

The most widely used test of this type was developed by OCE at its Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, N.H. (*3*) The CRREL test, based on heave rate, is more accurate than the indirect methods; however, it is a long and tedious test to perform and requires large-scale and expensive cold rooms and testing equipment. Many State highway agencies and some foreign countries have developed simpler, more rapid test methods to directly measure frost susceptibility. (*4*)[5] None of these methods has been adequately verified nor has the CRREL test been thoroughly verified by field data comparison studies, thus the need for full-scale field tests is well established.

The frost susceptibility value provides an order ranking for a particular soil and must be considered with the other physical and environmental factors to predict a pavement's performance in seasonal frost areas.

Status of Current Research Efforts

In the past 10 years several advancements have occurred in the determination of frost susceptibility and analytical modeling of frost action. Significant developments in laboratory and field test equipment and instrumentation systems have contributed to these advancements. But the lack of well-documented field data has delayed efforts to determine the best method to use, resulting in confusion and indecision for both designers and construction engineers (fig. 5).

The most significant development occurred in September 1978 when a comprehensive, 5-year research investigation was initiated by FHWA, FAA, and OCE to validate and refine certain selected frost action predictive

[4] "Materials Characterization of Thawed Subgrades," by M. W. Witczak. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

[5] "Rapid Test for the Determination of Frost Susceptibility of Soils," by R. M. Leary. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

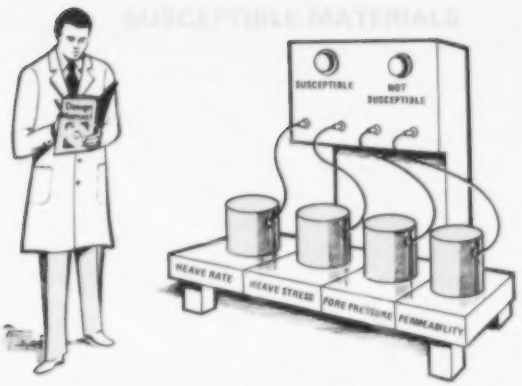


Figure 5.—Criteria for identifying frost-susceptible soils are greatly needed.

techniques. The investigation is being performed by CRREL and includes evaluation of the various devices for measuring frost susceptibility, mathematical modeling, laboratory predictive techniques for assessing strength loss due to thawing, and monitoring full-scale field test sites to provide correlative data.

Phase 1

Phase 1 of the CRREL study involves evaluating and selecting the best test method for determining the frost susceptibility of soils quickly and inexpensively. Three methods will be selected and validated with field test data. Each of the three chosen methods will be used to test six to eight soils that are being studied at the field test sites.

Phase 2

Phase 2 involves the mathematical modeling of frost action and deals with frost heave and thaw weakening. A preliminary computer model of frost heave was developed; an attempt will be made to develop a thaw weakening algorithm to be included in the mathematical model of frost heave.

The existing CRREL finite element model couples heat and moisture transport in freezing soil-water systems and provides quantitative predictions of the frost heave a soil will experience under different moisture and climatic conditions. The model includes algorithms for phase change of soil moisture as well as frost heave and permits several types of boundary and initial conditions. The Galerkin method of weighted residuals was used to simulate the spatial aspect and the Crank-Nicolsen method was used for the time domain portion of the finite element model. (5)

Preliminary verification of the frost heave model was conducted with the aid of laboratory soil column studies and also by comparing model predictions with available field data. Verification and refinement of the model under the current study will expand the laboratory soil column studies (Phase 3) and compare the predicted values generated from the model against field data obtained from Phase 5. The field data used in the preliminary verification phase were obtained from other sources and did not completely suit the study's needs.

The preliminary model was also inconclusively evaluated with respect to the effects of surcharge and overburden. These effects will be more fully evaluated under the present study by the use of the more elaborate soil column device designed and built in Phase 3.

Phase 3

In Phase 3 a laboratory soil column device will be designed and built to nondestructively measure moisture content, density, and soil suction changes that occur during unidirectional freezing of the soil sample. The device will be used to characterize parameters and evaluate and refine the mathematical models developed in Phase 2.

Although expensive to build and operate, the dual gamma energy device will provide greater accuracy, flexibility, and speed of testing because it simultaneously monitors key parameter changes without having to stop a particular test to take a moisture and density sample. The device should more than pay for itself in reduced testing costs during the life of the study.

As previously mentioned, the mathematical model was developed in terms of energy requirements to make water and heat move through soil. The water movement portion of the model is based on Darcy's Law, which requires knowledge of the hydraulic conductivity (coefficient of permeability), and the driving head, which is the sum of the pressure head and position head. Because pressure head can be positive or negative, piezometers and tensiometers will be included in the soil column to measure the pressures that are involved.

Another important input parameter is a diffusivity term, which is a function of the hydraulic conductivity and the slope of the moisture characteristic curve of the soil in question. The interaction of all of these properties of the soil-water system can best be studied by nondestructively monitoring the changes that occur as

the freezing front advances in the controlled environment of a dual energy soil column device.[6]

The device will consist of two gamma sources (Cesium will be used to measure moisture content changes and Americium will be used to measure density changes), detectors, signal conditioning and analyzing equipment, recording equipment, and a controller. The controller will permit automatic operation of the system or permit the operator to manually choose the source, vertical position, or counting time, as the situation dictates. In the manual mode the operator may record output from the system manually or have the output recorded on cassette tape. A movable stage is also provided to simultaneously move the radioactive (gamma) sources and their respective detectors vertically.

Phase 4

Phase 4 of this study is concerned with the development of a thaw weakening index of subgrade supporting capacity. This includes field and laboratory testing and is closely connected to the analytical work in Phase 2, which involves the development of a thaw weakening algorithm for the mathematical model of frost action.

The field investigations (Phase 5) include repeated-load plate-bearing (RPB) tests to determine the supportive capacity of subgrade or unbound base course soils during the frozen, thawed, recovered, and partially recovered conditions at the various test sites. Recoverable and nonrecoverable vertical displacements are measured at the plate itself and at various radial distances from the plate under a selected plate load.

FHWA research personnel performed RPB tests similar to the CRREL tests using the FHWA "Thumper." (6) The RPB and Thumper tests are being conducted at a load duration and frequency that represent a vehicle traveling at creep speed. The repetitive load triaxial tests are being conducted using the same load rate as the field tests. The duration and frequency of load application were selected so that no drainage would occur during loading, thus excess pore pressures would develop as under traffic loading.

The principal laboratory investigation for this phase of the work involved repeated load triaxial compression tests on undisturbed samples of the test site soils. The samples were obtained by coring the frozen subgrade soils during the first winter cycle of the test program. The samples remain frozen until tested and, upon completion of the repeated load tests, the samples are allowed to thaw and then are retested in the same manner. The test loads are stopped before the samples fail so that they remain suitable for retesting. The next tests are run on the same samples at different stages of recovery from thaw weakening. Thaw recovery is artificially produced by desaturating the thawed specimen to various partial saturation points. The CRREL researchers monitor moisture content, density, and stress state variables including moisture tension. The deformation moduli will be expressed in terms of these variables.[7]

Various resilient moduli for the soil at various depths in each test section were analyzed to determine which values gave surface vertical displacements that matched the measured field values obtained with the RPB equipment. The CHEVRON computer program (elastic layered systems analysis) and the FHWA VESYS system were used to calculate the vertical displacements.

Phase 5

Phase 5 involves two field test sites used by CRREL researchers to verify the analytical and laboratory predictive techniques. The Massachusetts test site is located in Winchendon at an off-road site on State Highway Department property behind a district maintenance depot, and the New York site is located at the Albany County Airport.

The Winchendon site was constructed in 1977 by the Massachusetts Department of Public Works (MDPW) to evaluate an improved test apparatus that was developed for MDPW by Massachusetts Institute of Technology (MIT) to determine the frost susceptibility of soils. The new device, the Freezing Soil Heave Stress System (FSHSS), was delivered to the MDPW in 1974 for in-house evaluation. Figure 6 shows typical test results. Although the FSHSS test results were reproducible, they did not correlate with CRREL heave rate test results. Because of the lack of field test results to validate either of the methods, it became necessary to conduct the additional field studies at Winchendon.

To minimize site-associated difficulties it was decided that a number of different soils would be studied under the same environmental conditions by importing

[6] "Mathematical Modeling of Simultaneous Transport of Moisture and Heat in Soils," by B. J. Dempsey. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

[7] "Proposed Investigation of Thaw Weakening of Subgrade Soil and Granular Unbound Base Course," by T. C. Johnson. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

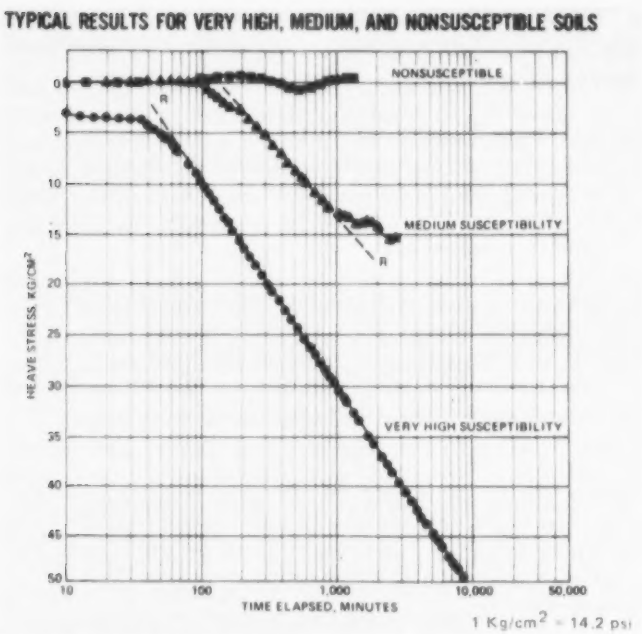


Figure 6.—Typical heave stress versus log time plots.

selected problem soils to the Winchendon site and placing them in prepared trenches (cells). The Winchendon site was chosen because it has a high natural groundwater table, granular subsurface soil with a relatively high permeability, and a relatively deep frost penetration.

Twelve different soils were obtained from various parts of Massachusetts and several neighboring States. Each soil was placed in a cell at two different water table elevations (0.9 and 1.5 m [3 and 5 ft]) and capped with a 75 mm (3 in) bituminous surface course. Figure 7 shows a partially completed soil test cell on the left and a completed cell on the right.

The data being collected in the field tests are as follows:

- Precipitation.
- Ambient temperature.
- Depth of frost penetration.
- Water content profile.
- Moisture tension.
- Frost heave.
- Subsurface temperatures.

In addition to the above data, RPB tests will be run by CRREL and FHWA during each seasonal variation to

Figure 7.—Each of the 12 soil types tested at the Winchendon site was placed in separate cells at two different elevations above groundwater level.

obtain appropriate pavement response data (deflections) under certain selected loads. As previously mentioned, undisturbed samples have been taken for use in the laboratory characterization program at the CRREL facility. Frost susceptibility and other soil tests not requiring undisturbed samples will be performed on soil samples gathered from stockpiles at the Winchendon site.

The Albany County Airport site, under observation since the fall of 1979, involves the construction of an extension to an existing taxiway and an existing but little-used taxiway pavement that has experienced detrimental frost effects.

The cross section of the extension will comprise 330 mm (13 in) of asphaltic concrete, 584 mm (23 in) of crushed stone base, and approximately 305 mm (12 in) of a granular subbase. The pavement cross section of the little-used taxiway is 102 mm (4 in) bituminous aggregate base and 406 mm (16 in) of granular base. The subgrade varies among a silty sand, silt, and clay at both sections.

The sampling and testing programs for the Albany site are very similar to those for the Winchendon site, except that sampling and instrumentation programs will be complicated because no construction work is planned for the little-used taxiway and because of the traffic on the existing taxiway.

Test pits were excavated along the little-used taxiway to obtain bag samples of each layer of soil and base material and to install instrumentation. The installation of instrumentation and collection of bag samples for the existing taxiway took place during construction operations. Borings were also taken during the first winter to obtain frozen samples of the subgrade soils to be used in the laboratory triaxial testing program.

Phase 6

Phase 6 involves formulating preliminary design and construction guidelines for pavements in seasonal frost areas. These guidelines will be prepared for practicing highway and airfield pavement design engineers. The preliminary guidelines will be expanded by CRREL under a separate contract with the Implementation Division of FHWA's Office of Development.

Potential Solutions to Frost Action Problem

Potential solutions may be classified into three main groups that will address the previously mentioned three major factors that control frost action in and beneath pavement systems—freezing temperatures, access to water, and frost-susceptible soil. All three factors must be present simultaneously to cause frost heave and thaw weakening problems; eliminate any one of them and you remove the problem. If the severity of any or all three of the factors is reduced, a corresponding reduction in frost action severity will occur—usually on about a one-to-one basis.

For example, freezing temperatures can be excluded or significantly reduced by using thermal barriers or insulators. The most popular insulating materials are extruded polystyrene boards (fig. 8) and foamed-in-place polyurethane. Many public works agencies have used this approach successfully; however, some have experienced serious problems with differential surface icing conditions between insulated and uninsulated pavement sections. Proper design of transition sections is required. (2)

Moisture barriers have also been used to cut off the supply of subsurface water that can migrate to the freezing front. This is another case of one theoretically sound solution creating another problem—the use of a capillary cutoff layer below the freezing zone but above the water table will trap infiltrating surface water between the pavement and the impervious layer.

Figure 8.—Placing polystyrene boards on a soil subgrade to provide a thermal barrier.

The most popular method for treating frost action involves removing or modifying the frost-susceptible soil element. Perhaps the best general source of information on this topic and the frost action problem in general can be found in reference 2.

In most cases public works agencies will control frost action damage by specifying nonfrost-susceptible material beneath pavements within the frost zone. For example, the MDPW specifications for soil and unbound base course materials within the frozen zone are as follows:[8]

- When the in situ soil within the frost depth contains more than 12 percent by weight passing the No. 200 sieve (0.074 mm [0.0029 in] opening) it shall be excavated and replaced with special borrow containing less than 10 percent passing.
- Embankment material within the frost depth shall consist of borrow material with less than 10 percent by weight passing the No. 200 sieve.

As previously mentioned, these specifications have proven to be very effective in excluding frost-susceptible soils; however, they are very expensive. One way to reduce the costs is to improve the accuracy and speed of frost susceptibility measurement equipment so that nonfrost-susceptible soils are not classified with the "bad actors" under the same specification that is based on particle-size classification.

A rapid test method is necessary to eliminate long delays whenever an unexpected change in the soil profile is uncovered during construction operations. The test should also be simple and inexpensive; many public works agencies will not invest in large-scale and expensive cold rooms and testing equipment.

Solution Selection

Collective efforts to solve the frost action problem have produced many new developments and many refinements of older processes or materials. However, the interaction of the soil element and the various environmental factors is not thoroughly understood. Until it is reduced to solvable mathematical expressions,

[8] "Full Depth Testing of Frost Susceptible Soils," by D. C. Rice. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

it will not be possible to rationally select from the available solutions.[9]

The proper solution may involve a combination of the various techniques that are available or are being developed. What is needed then is a rational decisionmaking methodology that includes appropriate tests and predictive techniques for assessing the potential for and consequences of frost action.

The mathematical model of frost heave potential should be a useful tool in selecting the proper course of action. This model will be especially valuable if an algorithm for thaw weakening can also be added to it.

The importance of the development and verification of an accurate, rapid, and inexpensive device or methodology for determining the frost susceptibility of soils is well established. It was previously explained to be both a design and construction control tool. The CRREL study will attempt to identify the best available tool and provide the required field measurements for verification. The carefully controlled and well-documented field studies will provide the necessary data base for improving any of the existing methods or the development of a newer, improved method. These data will be the first of their kind and will be valuable in the research community. (*4*)[10] [11] [12]

The development of a thaw weakening index of subgrade supporting capacity will also be a valuable design tool. This engineering approach to thaw weakening will supplement the scientific approach of the mathematical modeling phase. The empirical index will probably be based on the observed relationships between the resilient modulus and one or more of the following parameters:[13]

- Moisture tension.
- Moisture content.
- Density.
- Temperature.
- Permeability.
- Confining stress.
- Poisson's ratio.
- Various indices of material type.

The success of this work task depends on the correlation between the laboratory triaxial test results, the field RPB test results, and the soil column results. It also depends on the capability of the frost heave model to predict heave and changes in water content and density in the field.

The above discussion shows that it is not possible at this time to select the right "solution" to the "problem" of frost action beneath pavements. This is not to say that current design techniques are based on "guesswork" but rather that they are too conservative. The CRREL study is expected to provide answers that will form the basis for an improved design procedure for pavements in seasonal frost areas.

Summary

The damage to U.S. highways from frost action amounts to several billion dollars annually. Despite many years of research the problem has not been adequately defined, and therefore, rational solutions are unavailable.

FHWA is attempting to coordinate the resources of other Federal agencies and State highway departments to address this problem. The CRREL study is the major thrust of this effort, and a major breakthrough is anticipated.

Although each phase of the study is very important and each could stand alone, the most important aspect of the investigation is the soil column study. The results obtained from this portion of the investigation will tie together all of the other major aspects and provide the means for understanding the complicated frost action problem.

The analytical and laboratory testing programs that support the field testing programs are expensive, and the field testing and instrumentation monitoring costs are also high. It is unlikely that any Federal or State agency could afford to take on this challenge alone. Thus, the need to coordinate and combine resources is well established.

The results of the Winchendon and Albany field studies will provide valuable verification data; however, the resultant data base may be only marginally adequate. A

[9] "Mathematical Modeling of Simultaneous Transport of Moisture and Heat in Soils," by B. J. Dempsey. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

[10] "Rapid Test for the Determination of Frost Susceptibility of Soils," by R. M. Leary. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

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[13] "Proposed Investigation of Thaw Weakening of Subgrade Soil and Granular Unbound Base Course," by T. C. Johnson. Unpublished report presented at the FCP Research Review Conference, Atlanta, Ga., 1977.

series of satellite field studies in various parts of the Northern United States must be developed to validate and regionally adjust the findings of the CRREL studies at the two northeastern sites. Support and technical guidance will be provided to research personnel who are interested in conducting this type of study.

The use of physical models (full-scale field tests) to validate theoretical and laboratory models remains the most popular verification tool used by the highway industry. To be effective, these field testing programs must be planned, coordinated, constructed, and monitored as closely as possible with the way the predictive techniques were developed. The same assumptions must be used and all variables that cannot be controlled must be measured. If a series of field tests is to be conducted by various agencies at different locations, special care must be taken to insure uniformity of purpose and scope.[14]

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Albert F. DiMillio is a geotechnical research engineer in the Soils and Exploratory Techniques Group, Materials Division, Office of Research, Federal Highway Administration. Mr. DiMillio is project manager for FCP Project 4D, "Remedial Treatment of Soil Materials for Earth Structures and Foundations," and FCP Project 4H, "Improved Foundations for Highway Structures." Prior to his present position, he served as an area engineer in FHWA's Indiana Division Office and as the Regional Geotechnical Specialist in Region 5.

Donald G. Fohs is Chief of the Soils and Exploratory Techniques Group, Materials Division, Office of Research, Federal Highway Administration. He was formerly the project manager for FCP Project 4D, "Remedial Treatment of Soil Materials for Earth Structures and Foundations." Mr. Fohs joined the Materials Division upon graduation from the Junior Engineer Training Program in 1962. He has been active in soil stabilization research.

Development and Testing of Advanced Control Strategies in the Urban Traffic Control System

by
John MacGowan and Iris J. Fullerton

This is the last of three articles that trace the evolution and accomplishments of the Urban Traffic Control System (UTCS) research project. This series of articles highlights the research activities undertaken in the past 10 years and stresses the potential application and anticipated benefits accruing from this major research project. The first article, published in *Public Roads,* vol. 43, No. 2, September 1979, described the development of offline signal timing programs, the provisions of the real-world fully instrumented test laboratory established in Washington, D.C., and the first generation control strategy. The second article, published in *Public Roads,* vol. 43, No. 3, December 1979, summarized the hardware and strategies of the Bus Priority System (BPS), the second and third generation software systems, the evaluation of the various strategies, simulation model development, and vehicle detection. This article presents the technical accomplishments of the UTCS research project.

Introduction

The primary purpose of the UTCS project was to generate knowledge and experience in the design and implementation of computer-based traffic control systems. The preceding articles of this series detailed the activities undertaken within the UTCS project. The following are the major results of these activities and their application to traffic control in urban areas.

The UTCS project makes available significant data on planning digital computer-based traffic systems. The advantages to the users are as follows: Technical information that assists the explicit preparation of design specifications; successful system procurement by specifying proven software; easier staged construction without tying the agency to the original contractor; better system changes nationwide; reduction of procurement costs by eliminating the costs of new commercially developed software; and an interchange of operators among similar systems in various cities.

In addition to these advantages associated with the overall planning and implementation of a computer-based system, the UTCS system provides significant operational advantages to the user agency. For example, the UTCS system provides the following:

- Capability to use many signal timing plans designed for special problems or periods of the week.
- Operational flexibility because of operator-in-the-loop functions.
- Immediate awareness of operating status using a map display or other available display devices.
- Surveillance information for recurring traffic problem locations.
- Instant notification and diagnosis

of system element malfunction to facilitate hardware maintenance.

- Automatic data recording for engineering analyses.
- Computation of measures of effectiveness that may be correlated with other contemporary measures such as fuel consumption, vehicle emissions, or safety.

Results of the Development of Advanced Strategies

The research results are summarized in perspective. First generation control (1-GC) improved operations in Washington, D.C., in the phase I evaluation; in the phase II evaluation, second generation control (2-GC) performed approximately equal to a well-timed pretimed system. Third generation control (3-GC) was disappointing when compared to the well-timed pretimed system. Critical intersection control (CIC) strategies did not improve overall system performance for any system with which it was evaluated. However, net benefits were derived during certain periods of the day by judicious application of CIC. Bus delays were reduced at intersections by granting preferential treatment. A preemptive scheme of allowing additional green time to buses until they cleared the intersection approach or granting an early green signal indication worked well when properly designed for existing conditions.

Summary of findings—phase I

The results of the 1-GC alternatives evaluation are summarized in table 1. (*1*)[1] The traffic responsive (TRSP) alternative generally matched or exceeded the performance of the other traffic control alternatives. The differences in traffic delays were relatively small, ranging from 2 percent better than TRSP to 4 percent worse. These small differences may have resulted because the three time periods tested were not truly representative of the total potential for improvement. This indicates that longer periods of the day should be considered in evaluations.

Table 1.—Summary of performance of tested strategies—phase I

Comparison[1]	Time period 7:00–9:45 a.m.	1:00–3:00 p.m.	4:00–6:30 p.m.
TRSP versus DCP	−4.0	+0.3[2]	−3.9[3]
TRSP versus TOD	−1.8	−2.8[3]	+2.0
TRSP versus CIC	+0.7	−0.5	−2.8[4]
TRSP versus BPS	−2.5	−0.8	−0.3

[1] In terms of aggregated delay.
[2] The overall delay reduction in the dense grid portion of the network: the total increase in delay in other portions of the networks. A positive sign indicates less delay with alternative when compared with TRSP.
[3] Statistical significance at 5 percent level.
[4] Statistical significance at 1 percent level.

Despite little difference in performance, the computer-based system remains an attractive alternative. Fewer person-hours are required to develop the timing using a computerized optimization program than are required for manual preparation of timing. Also, the computer-based system has greater operating flexibility and improved hardware maintainability. In addition, the effectiveness of user-generated signal timing plans is site-specific. The impact of these plans on traffic flow efficiency may vary considerably, depending on the efficacy of the original plans to which they are compared.

The phase I evaluation also indicated that the CIC alternative equally distributed delay on the approaches to a given intersection. Under TRSP, approaches with good offset relationships and low delay were generally degraded by the CIC algorithm. Approaches with poor offset relationships and high delay were improved. CIC can be more useful on arterials than in grids because of the fewer offset and capacity conflicts.

The BPS algorithm worked well; bus delays were reduced as much as 42 percent (from 0.831 to 0.481 minutes) on a 1.5 km (0.93-mile) section of a given route. There is a level of bus activity beyond which the BPS operation would be unstable. More than 30 to 50 buses per hour or nearly 1 bus per cycle would lead to congestion at the BPS intersection. Automobiles, as well as buses, would be penalized by the intersection congestion, and the BPS algorithm should be switched off. At the higher bus levels, fixed time plans could be developed to account for constant bus presence.

Summary of findings—phase II

The results of the comparisons between the advanced control strategies (2-GC and 3-GC) and the base case (Washington, D.C., timing plans [DCP]) during phase II are given in table 2. (*2*) Differences in the performances of the strategies are based on a direct comparison of vehicle-minutes of travel in the

[1] Italic numbers in parentheses identify references on page 165.

Table 2.—Summary of performance of tested strategies—phase II

Comparison[1]	Time period				
	8:00–9:45 a.m.	10:15–1:00 p.m.	1:00–3:45 p.m.	3:45–6:15 p.m.	7:00–8:00 a.m. 9:45–10:00 a.m. 6:15–7:00 p.m.
2-GC versus DCP	+3.5[2]	+3.2	+1.1	+9.3[3]	+1.9
3-GC versus DCP	+13.4[3]	+9.8[3]	+3.4	+9.1	+15.2[3]

[1] In terms of vehicle-minutes of travel.
[2] A positive sign indicates greater vehicle-minutes of travel when compared to DCP.
[3] Statistical significance at 5 percent level.

network.[2] When two data sets are compared, the vehicle-minutes of travel represent a combination of travel time and volume, resulting in a "total travel time difference."

The 2-GC control strategies were the first of the advanced control algorithms to be tested. Although networkwide it showed 4 percent degradation in performance relative to the DCP alternative, 2-GC performed better than all other alternatives on the arterial portion of the network showing approximately a 2 percent improvement.

When evaluated over a 10-year period, estimated costs for using 2-GC are 10 to 15 percent higher than for 1-GC. After 10 years, the costs become essentially equal. Given the relatively small cost difference between 2-GC and 1-GC and the self-adapting characteristics of 2-GC, 2-GC offers promise and warrants further experimentation on its application and effectiveness.

During all time periods, 3-GC was less efficient than DCP; 3-GC experienced approximately 10 percent more vehicle-minutes than DCP. Its performance was comparable to DCP in only one case (mid-afternoon, in the dense grid portion of the network). Possible explanations of 3-GC's poor performance relative to the other strategies include the following:

- 3-GC depends on good surveillance and historical traffic data, and they may not have been accurate.
- The saturated network portion of 3-GC was not tested. The unsaturated portion was not intended for congested flow operations and, as expected, performed poorly during these conditions.
- There was not enough calendar time available after 3-GC development to permit the necessary fine tuning and calibration.

The conclusion is that 3-GC must still be considered in the "research and development" stage, and the basic concept of variable background cycle networkwide should be reexamined.

Summary of findings—New Orleans

The evaluation results of the UTCS 1-GC system implemented in New Orleans, La., are presented in table 3. (3) This table compares strategies for each time period for the total network. The comparison of New Orleans timing plans (NOP) with time of day (TOD) indicated a statistically significant 8.8 percent improvement of TOD over NOP. The comparison of NOP with TRSP indicated a statistically significant 8.5 percent improvement of TRSP over NOP. A negligible 0.2 percent difference resulted from a comparison of TOD with TRSP.

These results show a more substantial and consistent improvement in traffic flow in New Orleans with UTCS computer control than in traffic flow in Washington, D.C., with UTCS. The direction of change is generally consistent for UTCS 1-GC evaluations, but the improvement was greater in New Orleans.

Given that the procedures were similar, the differences are probably related to the less congested network conditions found in New Orleans. Under these conditions, the traffic signal system has a more direct impact on traffic flow. Additionally, it is likely that the DCP plans more closely matched existing conditions than did the NOP plans. The New Orleans results are probably typical of cities with populations less than one million because the base signal system and traffic characteristics are

[2] The comparison between DCP and TRSP is given in table 1. That comparison is given in terms of aggregated delay.

Table 3.—Summary of performance of tested strategies—New Orleans, La.

Comparison	Time period 7:00–9:00 a.m.	9:00–11:30 a.m.	11:30–1:45 p.m.	1:45–3:30 p.m.	3:30–6:30 p.m.	6:30–9:00 p.m.	7:00 a.m.–9:00 p.m.
TOD versus NOP	+15.5[1]	+ 9.8	+ 9.1	+2.5[2]	+7.9[2]	+ 8.7	+8.8[2]
TRSP versus NOP	+18.4[2]	+15.2[2]	+10.8	+2.7	+0.4	+13.2	+8.5[2]
TRSP versus TOD	+ 2.9	+ 5.5	+ 1.7	+0.2	−7.5	+ 4.5	−0.2

[1] A positive sign indicates less vehicle-minutes of travel with the first alternative when compared with the second alternative.
[2] Statistical significance at 5 percent level.

more typical than those found in Washington, D.C.

Results of System Design and Specification Research

The system design and specification phase of the research project provided much information and contributed significantly to the development of techniques that are used today. The hardware and software configurations used for the UTCS test area may be applied, in whole or in part, to any urban traffic control system. The size and complexity of the system installed in the test area required additional detectors and computer data handling for evaluation. The UTCS software and its data base, however, can be scaled to suit each local application. The programing of the UTCS software in FORTRAN language provides the maximum flexibility to use various computers as well as the minimum programing for machine-related routines and data base requirements. It also can accommodate future modifications and updates.

When implementing the system, the number of required detectors and controllers and the data handling capability of the central data processor would depend on the number of intersections to be controlled and the local traffic characteristics. The peripheral equipment at the data processing site and the sophistication of the software package can be tailored to reflect data collection needs and budget limitations. UTCS results should be adapted to suit the particular application.

Preliminary surveys and studies of the local area would be needed to derive a street installation design providing the maximum benefits. However, the street installation design affects certain aspects of the central design including the built-in growth capability of the computer configuration. The network size, communication technique selection, and logic signal levels for interfacing through the computer interface unit (CIU) have considerable impact on the communications subsystem design. The CIU must be designed to accommodate the computer-generated commands for the total number of planned intersections and any special functions such as signal preemption. In addition, it must accommodate logic signals to computer peripheral equipment and other control and display devices such as the map display. (*4*)

Although the basic UTCS map display design can be utilized—including the symbols, legends, and lighting—the actual map, the number of displays, and the scaling must be appropriate to the locality.[3]

Results of Research in System Operation and Maintenance

Because objectives of the UTCS research project included providing guidelines and disseminating information to prospective users, much attention was given to the collection of operations and maintenance data. The following are some of the relevant aspects of the UTCS/BPS system operating experience from November 1972 to June 1976. (5)

A typical operational system should be readily accommodated in an approximately 110 m² (1,200 ft²) area. If possible, the control center room should be isolated from building entrances by partitions or separated rooms to avoid severe temperature variations and dust accumulation. The control center should be separated from any office facilities and located in a controlled access area to keep out unauthorized personnel. A fire protection system

[3] Map displays are discussed in detail in "Map Displays for Traffic Systems," by D. Sundberg, *Public Roads*, vol. 42, No. 1, June 1978, pp. 6–16.

for the computer system components should be part of the control center design. Also, acoustical design factors should be used to minimize ambient noise; air conditioning units should be located outside the center to minimize noise.

Each unit of peripheral equipment should have an individual circuit breaker, and where power outages frequently occur, a backup power source should be provided.

Continuous operation of a system requires onhand supplies of expendable materials (tapes, cards, paper) and various stored program materials (disks, tapes), which should be conveniently stored.

The UTCS control panel was useful in determining system and equipment status and in facilitating maintenance activities. Similarly, a dynamic map display aided in troubleshooting the system, debugging and refining control techniques, and evaluating performance.

Responsibilities should be clearly designated to a specific individual or position for both operations and maintenance. It should be the operations manager's responsibility to establish and enforce procedures for emergency situations, special traffic situations, coordination with police, correcting malfunctions, alerting appropriate personnel of system status, and incorporating system changes. Another important role of the operations manager is communication with the media (press, television, radio) to keep the public informed of the system's status and improvements and to aid motorists. A staff of three people can adequately handle a normal, single-shift operation (peak-to-peak attended). A system operator can program, and a field maintenance technician can carry out normally required functions. The third person can serve as a backup in various troubleshooting activities.

The maintenance manager is responsible for systematic identification and isolation of malfunctions, direction of repair, parts procurement, and vendor warrantee processing. Other necessary tasks in effective maintenance management include arranging the required services, monitoring and verifying their completion, validating invoices, and updating as-built drawings.

The system operator should maintain a log of all system activities for tracing the operational performance history and maintenance and troubleshooting. A procedure that keeps the traffic engineering and maintenance personnel informed of all changes is essential.

It should be recognized that a computer-controlled signal system will have more stringent requirements and deployed equipment than a conventional system. Proper consideration should be given to budgeting for this.

The key factor of adequate maintenance staffing is the early assignment and training of personnel. Negative effects of turnover in staff are alleviated by good replacement training and system documentation.

The reliability of equipment and turnaround time for repairs should be projected. A repair strategy for field equipment must be established. In UTCS, when field repairs were necessary, a new unit replaced the faulty one in the field. The faulty unit was returned to a depot for repair or cycled back to the supplier for warrantee servicing. It was found that many units could be repaired by replacing a unit component. This resulted in significant cost savings and required fewer unit and module spares.

One item considered essential is a two-way communication link between the field and the control center to evaluate operation and performance. Both radio and voice line to intersection techniques are available but in any case, a communication link is a cost-effective and timesaving aid in troubleshooting a system.

System Costs and Benefits

Comparing system costs is difficult because of the variability in procurement, the extent of implementation activities, and the amount of recurring operations and maintenance costs including the following: Operating staff costs, maintenance staff costs, service contract costs, and replacement hardware costs.

As a result of the UTCS research conducted over the past 10 years, approximately 150 U.S. cities are in some stage of implementing UTCS or similar systems. Several cities have determined that installing the systems reduces traffic congestion and increases fuel efficiency. Table 4 shows these benefits for three medium-sized cities and relates the benefits to the time required to recover installation costs.

The New Orleans, La., system used UTCS; the Greensboro, N.C., system used a software package functionally equivalent to UTCS; and the Raleigh, N.C., system used a software package similar to UTCS. Fuel savings alone will recover installation costs within a 10-year lifespan of these systems (table 4). When savings from reduced congestion are considered, the installation costs can be recovered in 1 to 2 years.

The variability in system costs makes it difficult to extrapolate these data for other cities. However, it should be noted that the cost of the computer complement combined with

Table 4.—Example benefits from three medium-sized cities *(3, 6, 7)*

City	Number of intersections	Installation costs	Benefits per year[1] Reduced fuel consumption		Reduced person-hours	Travel time cost savings	Total cost savings	Years to recover investments[2] Fuel savings only	Travel time savings only	Total
		Millions	*Gallons*	*Millions*	*Thousands*	*Millions*	*Millions*			
New Orleans, La.	202	$4.40	926,000	$0.93	1,924	$5.10	$6.03	4.7	0.9	0.7
Greensboro, N.C.	159	1.35	360,000	0.36	260	0.50	0.86	3.7	2.7	1.6
Raleigh, N.C.	153	0.66	286,000	0.29	364	0.80	1.09	2.3	0.8	0.3

[1] Based on actual field data and the following conversion factors: 1 gal=3.8 l
1.3–1.4 persons per vehicle.
260 work days per year.
Cost of fuel $0.95–$1.05 per gallon.
Value of time $2.00–$2.65 per hour.

[2] Does not include recurring costs, benefits from enhanced operations and maintenance. Life-cycle of these systems is figured at 10 years, although many conventional systems with same life-cycle have been operating for more than 20 years. No attempt has been made to convert future savings to present dollar value.

programing is usually less than 25 percent of system cost.

Application Conclusions

The application conclusions of UTCS-type strategies and concepts in urban areas are summarized below:

- 1-GC (TRSP) is a viable software system and should be given primary consideration. 1-GC is also the least expensive to apply.
- 2-GC is a potentially effective online control strategy. It should be considered and tested on arterials and networks outside the Washington, D.C., UTCS site.
- 3-GC has the potential for improving traffic performance but needs further research.
- CIC proved effective with the 1-GC (TRSP) alternative at equalizing delays on competing approaches to individual intersections. Although the 1-GC CIC algorithm did not show consistent improvement networkwide, CIC can be used effectively at a limited number of intersections—particularly arterials because of fewer offset and capacity conflicts.
- BPS proved effective at locations insensitive to offset changes and where peak hour bus volumes average 30 or less buses per hour.
- Design concepts developed during the UTCS project provide guidance for system specification preparation and more successful system procurement.
- Operational flexibility is enhanced, allowing higher productivity of staff and budget.
- Maintainability—detection, diagnosis, and repair—is improved, providing motorist benefits from greater uptime for system components.
- Because of direct and indirect benefits to motorists, UTCS systems "pay for themselves" within their life cycles. System efficiencies and flexibility allow more effective control of traffic signals than was ever possible with conventional systems and at a greater savings to the public.

REFERENCES[4]

(1) J. L. Kay and J. C. Allen, "Evaluation of First Generation UTCS/BPS Control Strategy, Volumes 1, 2, and Executive Summary," Report Nos. FHWA-RD-75-26, -27, and -28, *Federal Highway Administration*, Washington, D.C., February 1975. PB Nos. 244111, 244112, and 244110.

(2) R. D. Henry, R. A. Ferlis, and J. L. Kay, "Evaluation of UTCS Control Strategies, Executive Summary and Technical Report," Report Nos. FHWA-RD-76-149 and -150, *Federal Highway Administration*, Washington, D.C., July 1976. PB Nos. 275044 and 275045.

(3) R. D. Henry, R. A. Ferlis, and R. M. White, "Application of UTCS First Generation Control Software in New Orleans, Executive Summary and Final Report," Report Nos. FHWA-RD-78-2 and -3, *Federal Highway Administration*, Washington, D.C., January 1978. PB Nos. 287359 and 287360.

(4) "Urban Traffic Control System Hardware, A Specifications Checklist," Implementation Package 76-1, *Federal Highway Administration*, Washington, D.C., February 1976.

(5) "UTCS/BPS Operations and Maintenance Final Report," Report No. FHWA-RD-76-160, *Federal Highway Administration*, Washington, D.C., June 1976. PB No. 276377.

(6) JHK & Associates, "Greensboro Traffic Control and Surveillance System Before and After Evaluation—Draft Final Report, Project 8.6070811," *North Carolina Department of Transportation*, Raleigh, N.C., December 1977.

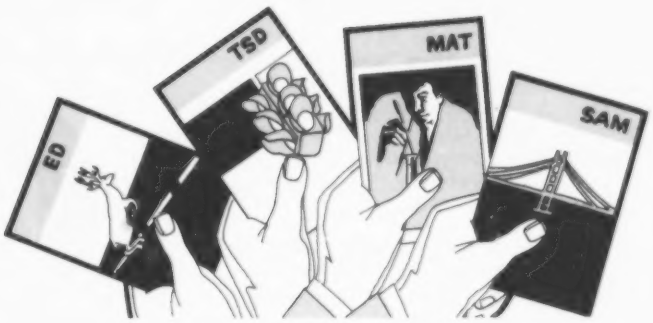
(7) JHK & Associates, "Raleigh Traffic Control and Surveillance System Before and After Evaluation—Final Report, Project 8.6052014," *City of Raleigh*, Raleigh, N.C., May 1977.

[4]Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

John MacGowan is a highway research engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been with FHWA since 1970, first as a staff member on the UTCS research team and then as the project manager of FCP Project 2B, "Development and Testing of Advanced Control Strategies in the Urban Traffic Control System." Mr. MacGowan is currently the project manager of FCP Project 2M, "Arterial Flow and Control." This research concerns arterial capacity, signal control for arterials, and bus signal priority systems.

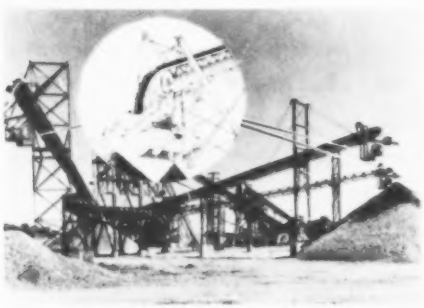
Iris J. Fullerton is a Senior Public Systems Specialist for JHK & Associates. She specializes in research synthesis, systems evaluation, engineering training, and the communication and implementation of applied research. She is currently principal investigator for the FHWA project "Roadway Delineation Practices" and is participating in the National Cooperative Highway Research Program (NCHRP) Project 3-28 to develop an improved Highway Capacity Manual. Ms. Fullerton participated in the evaluation of UTCS control strategies and NCHRP Project 3-18(3), "Cost Effectiveness Methodology for Evaluation of Signalized Street Network Surveillance and Control Systems." She coauthored *Urban Street Design*, the student textbook for the FHWA's Urban Transportation Operations Training Program.

Recent Research Reports You Should Know About



The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Division. The reports are available from the address noted at the end of each description.

Techniques for Automatic Aggregate Gradation, Report No. FHWA-RD-79-118



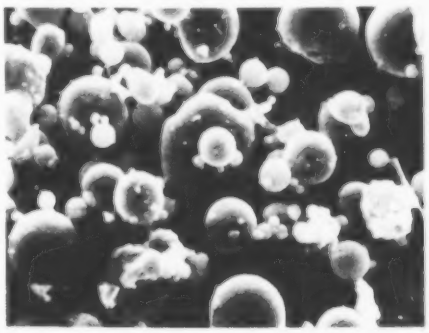
by FHWA Materials Division

This report evaluates techniques for automatic gradation of mineral aggregate used in highway construction. The report provides information on aggregate gradation for process control and producer certification of highway aggregate.

Six potential techniques for automatic particle-size identification were studied after an extensive review of industrial methods. Two of these six techniques—the vidicon and the optical shadow—were used to develop a conceptual system design. The design includes technique modification, system component identification, structural considerations, compatibility with existing plant equipment, system reliability analysis, cost estimates, and analysis of technique limitations.

No commercially available technique meets all the industry-specified criteria for automatic gradation, however, the vidicon and optical shadow techniques appear viable. Engineering model development and field testing are recommended to improve their performance.

The report is available from the Materials Division, HRS-22, Federal Highway Administration, Washington, D.C. 20590.



The Application of Brown Coal Fly Ash to Road Base Courses, Report No. FHWA-RD-79-101

by FHWA Materials Division

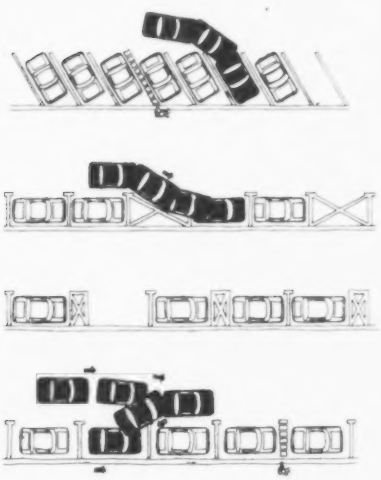
This report describes research performed at the Polish Ministry of Transport's Road and Bridge Research Institute. The research should be useful to highway construction and maintenance engineers in the Midwest and Southwest United States where increasing quantities of fly ashes from subbituminous and lignitic coals are being generated in power plants.

The research findings indicate that reactive brown coal fly ash can stabilize cohesive soils to improve subgrades for all-purpose roads and subbases for lightweight roads. Free lime, the reactive ingredient in these fly ashes, can range from 7 to over 20 percent by weight. Noncohesive soils can be stabilized using both reactive

and nonreactive fly ashes, with corresponding decreases in cement requirements. The report recommends investigating potential uses of nonreactive brown coal fly ash as a filler in portland cement concrete and bituminous mixes for road embankments.

The report is available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.

Safety Aspects of Curb Parking, Executive Summary (Report No. FHWA-RD-79-75) and Final Technical Report (Report No. FHWA-RD-79-76)



by FHWA Traffic Systems Division

These reports describe the effects of onstreet parking on safety and traffic operations. An analysis was made of over 4,800 accidents on 273 km (170 miles) of major, collector, and local streets in 10 cities with varying street widths, land uses, and types of parking. Field studies compared the effects of different types of parking on operating speeds, parking times, and pedestrian conflicts. The types of parking examined were no parking, conventional parallel parking, paired parallel parking, flat angle parking, and high angle parking. The study found that angle parking is no more hazardous than parallel parking when land use, parking turnover, and street type are considered.

The executive summary highlights the study findings and the final technical report details the research techniques and analyses.

Limited copies of the reports are available from the Traffic Systems Division, HRS-33, Federal Highway Administration, Washington, D.C. 20590.

Guidelines for the Application of Arrow Boards in Work Zones, Report No. FHWA-RD-79-58

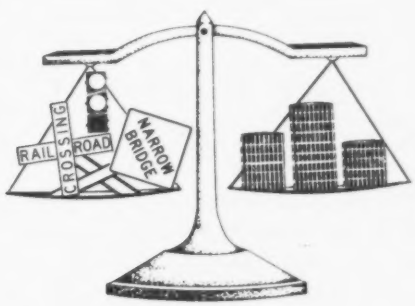
by FHWA Traffic Systems Division

This report contains guidelines for the application of arrow boards in highway work zones, including design specifications, warrants for use, and placement details. Human factors investigations were conducted in the laboratory to determine driver information requirements, expectancy, and understanding of arrow boards. Field studies of driver responses to arrow boards were conducted in 78 construction and maintenance zones in California and Illinois.

The research determined that arrow boards were effective in lane closure work zones when placed on the shoulder of the highway near the start of the lane closure taper because they prompted earlier merging into the open lane. Arrow boards were not found to be generally effective in traffic diversions or splits or for shoulder closures; however, they did reduce some specific operational problems in these types of work zones.

Limited copies of the report are available from the Traffic Systems Division, HRS-33, Federal Highway Administration, Washington, D.C. 20590.

Assessment of Techniques for Cost-Effectiveness of Highway Accident Countermeasures, Executive Summary (Report No. FHWA-RD-79-52) and Final Report (Report No. FHWA-RD-79-53)



by FHWA Environmental Division

These reports describe the accuracy, sensitivity, and usefulness of cost-effectiveness analysis methods for evaluating highway-related accident countermeasures. The executive summary highlights the findings in the five-part final report.

Part one contains a classification and discussion of available methods of cost-effectiveness analysis.

Part two reviews highway safety activities at the Federal, State, and local government levels.

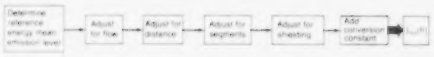
Part three evaluates the usefulness of cost-effectiveness submodels in identifying accident locations and estimating countermeasure effectiveness. The sensitivity and importance of accurate, reliable information on accident costs, highway user benefits, and costs of countermeasures are discussed.

Part four describes methods for determining countermeasure effectiveness and reviews the effectiveness and cost of seven countermeasures: Adding shoulders to two-lane highways; resurfacing pavements with inadequate friction; installing signals at a stop-controlled, simple intersection on a moderate-volume highway; installing flashing lights at railroad-highway grade locations; installing delineators on horizontal curves; installing impact attenuators at raised gore areas; and widening bridges.

Part five gives recommendations regarding cost-effectiveness analysis and recommends three specific cost-effectiveness methods—a dynamic programing algorithm, an improved incremental benefit-cost algorithm, and an integer programing algorithm.

The same techniques recommended for use by State and local governments can also be used to perform nationwide highway safety needs studies. To perform evaluations at the Federal level, accident data for selected locations must be collected from sampling data available in the States.

The reports are available from the Environmental Division, HRS-41, Federal Highway Administration, Washington, D.C. 20590.



Users Manual: FHWA Level 1 Highway Traffic Noise Prediction Model SNAP 1.0 (Report No. FHWA-RD-78-139) and Users Manual: FHWA Level 2 Highway Traffic Noise Prediction Model STAMINA 1.0 (Report No. FHWA-RD-78-138)

by FHWA Environmental Division

The SNAP 1.0 report describes the FHWA level 1 highway traffic noise prediction model. The model allows rapid calculation of highway traffic noise emissions for simple roadway-receiver configurations. Computed output is presented in tables for direct inclusion in reports.

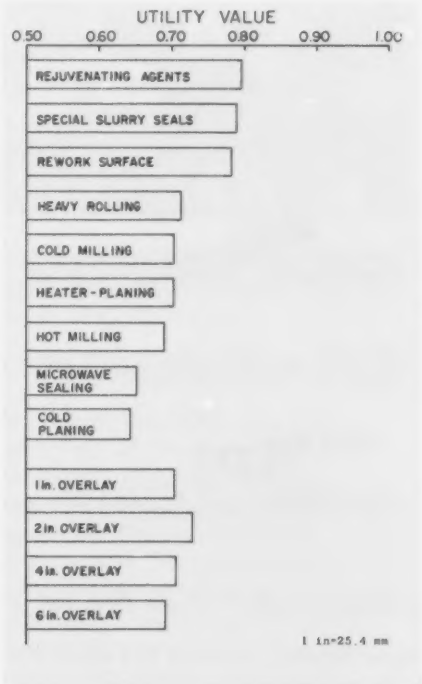
The report describes the formulation and format of input data and the predicted traffic noise estimates. Noise attenuation is considered for traffic lanes parallel to both "thin screen" and "berm" type barriers. Vehicle noise emissions are estimated by vehicle type and vehicle speed.

The STAMINA 1.0 report describes the FHWA level 2 highway traffic noise prediction model and modifications to the TSC MOD-04 highway traffic noise prediction program. The level 2 model extends the scope of problem formulation. It features revised vehicle reference noise emission levels; specification of site-specific excess attenuation; English, metric, and metric/English conversion of engineering units for both input and output data; common input data format with the TSC MOD-04 model; and user options to improve operating efficiency.

The report describes problem formulation, input data requirements, output error messages, examples of usage, and computer program documentation.

The reports are available from the Environmental Division, HRS-42, Federal Highway Administration, Washington, D.C. 20590.

Techniques for Rehabilitating Pavements Without Overlays—A Systems Analysis, Volume 1—Analysis (Report No. FHWA-RD-78-108) and Volume 2—Appendixes (Report No. FHWA-RD-78-109)



by FHWA Structures and Applied Mechanics Division

These reports identify 92 promising techniques for rehabilitating deteriorated pavements in place without using overlays. Sixty-two of these techniques are analyzed in detail. A utility value for each technique was established through

computerized decision analysis methodology, which considered 17 feasibility factors grouped into four categories: Costs, performance, energy implications, and safety or environmental impacts. Nineteen of the techniques analyzed exhibit excellent pavement rehabilitation potential and warrant field evaluation. The use of rejuvenating agents and the reworking of pavement surfaces were rated highly as flexible pavement rehabilitation techniques for particular pavement distress problems. Applying rejuvenating agents was most beneficial for flexible pavement surface loss and deterioration; surface reworking was more effective for pavement holes or indentations, profile distortion, and asphalt bleeding.

Two methods were recommended for repair or replacement of failed joints in rigid pavements. At joints over granular bases where spalling, faulting, or pumping is present, the pavement structure near the distress is strengthened by drilling a horizontal opening under each joint and backfilling with a strong, permeable material to provide a sleeper slab. The second method involves a new precast replacement joint assembly design that maintains load transfer along a single new transverse joint rather than along two joints.

Transferring traffic wheel load concentrations from distressed paths by laterally shifting the lane markings is a technique for extending pavement service life without structural change. This preemptive method may be used where deterioration has not progressed significantly and where the pavement is wide enough to permit such a remedy.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 297403 and PB 297404).

Extending the Service Life of Existing Bridges by Increasing Their Load Carrying Capacity, Report No. FHWA-RD-78-133

by FHWA Structures and Applied Mechanics Division

This report identifies typical structural deficiencies of older inservice bridges and presents new techniques for increased load carrying capacity of such bridges through repairs, strengthening, improved geometrics, and other upgrading measures.

One hundred and forty older, substandard steel, concrete, and timber bridges, ranging in span from 6 m (20 ft) to 60 m (197 ft) and located in five States, were inspected to categorize bridge types and deficiencies. Two classes of deficiencies are defined: Those resulting from poor design and those resulting from age and wear. Deficiencies are further cataloged as structural, mechanical, geometric, or safety related.

Bridge rehabilitation practices in nine States and innovative measures for upgrading the structural adequacy of existing bridges are reviewed and evaluated. The most applicable techniques were used to prepare example rehabilitation plans for five typical bridges. Estimates of rehabilitation costs for four of the five bridges are as low as one-tenth of the cost for span replacement.

The proposed structural modifications for increasing live load capacities fall into four general classes: Strengthening critical members, adding supplemental members, reducing dead loads, and modifying the structural systems. Innovative measures investigated include applying external reinforcing plates with adhesives, making a series of simple spans into a continuous span, and adding supplemental supports to reduce span length. Further development of other new and unique techniques is recommended.

Cost-effective measures to improve traffic safety and capacity are available for geometrically modifying all bridge types except through-truss structures.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 294508).

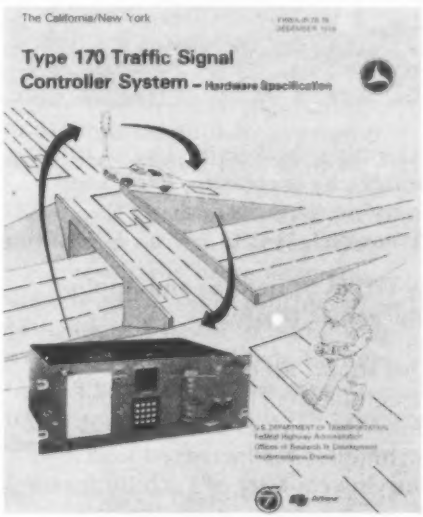
Implementation/User Items "how-to-do-it"

The following are brief descriptions of selected items which have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies.

U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division (HDV-20)
Washington, D.C. 20590

Items available from the Implementation Division can be obtained by including a self-addressed mailing label with the request.

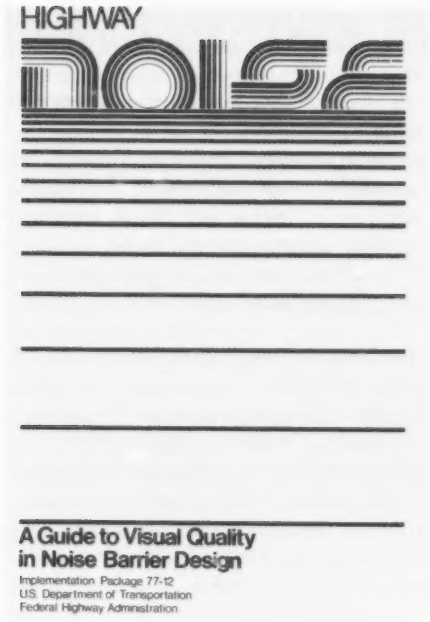
The California/New York Type 170 Traffic Signal Control System—Hardware Specification, Implementation Package 78-16



by FHWA Implementation Division

This report constitutes the completed hardware specification used by California and New York to purchase Type 170 traffic signal controller systems. California and New York purchased over 1,000 microcomputers, performed acceptance testing, installed software, and installed the microcomputers as intersection controllers. The concept reduced purchase cost and improved flexibility and expediency. California and New York have prepared three sets of software with appropriate documentation for the microcomputer-based controllers. This software will control two- to eight-phase intersections, diamond interchanges, and ramp meter locations. Diagnostic maintenance and acceptance testing programs to be used with the controllers are being developed.

Limited copies of the report are available from the Implementation Division.



A Guide to Visual Quality in Noise Barrier Design, Implementation Package 77-12

by FHWA Implementation Division

This guide explains how visual design principles can be incorporated into the design of highway noise barriers. The three current forms of noise barriers—earth berms, walls, and combinations of berms and walls—can create visual problems because of acoustical requirements. Potential visual problems and design measures that minimize the visual disruption are illustrated. This guide will aid highway engineers in designing noise barriers that are functional, attractive, and visually related to the environment.

Limited copies of the guide are available from the Implementation Division.

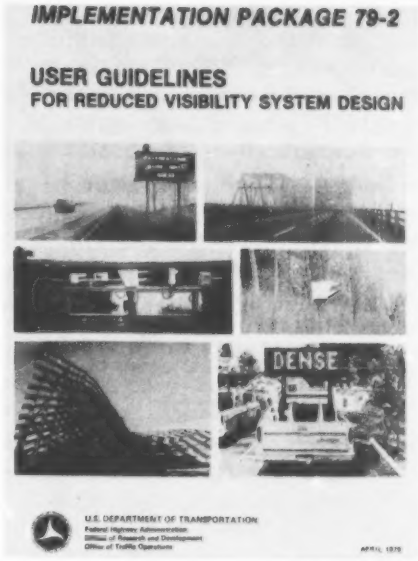
Highway Pollution Dispersion: Air Quality in the Right-of-Way

by FHWA Implementation Division

This 16 mm color/sound film examines pollutant dispersion within the highway right-of-way by presenting concepts of air flow patterns around moving vehicles, manmade structures, and adjacent landforms. The effects on dispersion caused by various highway cross sections including street canyons are also presented.

This 20-minute film offers specific research insights on air quality. An important point illustrated in the film is that vehicles emit substantial amounts of waste heat, creating an unstable air mass in the right-of-way. This artificially induced turbulence enhances the dispersion of air contaminants, resulting in lower concentrations in the near roadway environment.

The film is available on loan from the National Highway Institute.



User Guidelines for Reduced Visibility System Design, Implementation Package 79-2

by FHWA Implementation Division

This report contains guidelines for identifying locations with reduced visibility problems, quantifying the extent of the problem, establishing candidate systems, and estimating costs.

Reduced visibility guidance systems were evaluated to determine effective instrumentation, installation techniques, and operational measures. Information was obtained from many operational systems. The report includes information on system configurations, sensing and signing equipment, and communication techniques. The advantages and disadvantages of different equipment types are discussed, and recommendations are made for reduced visibility applications.

The report may be purchased for $4.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00146-2).

Seismic Retrofit Measures for Highway Bridges, Volume 1 (Report No. FHWA-TS-79-216) and Volume 2 (Report No. FHWA-TS-79-217)

by FHWA Implementation Division

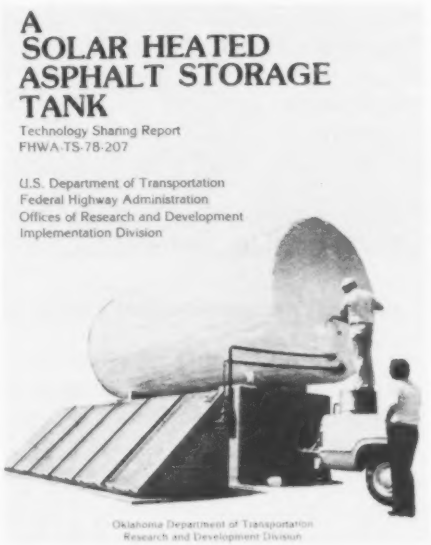
Most highway bridges in the United States have not been designed to resist seismic forces. Since the 1971 earthquake in San Fernando, Calif., highway engineers have been concerned about the potential for earthquake damage to bridges. Bridges are being evaluated to determine if they warrant protection against seismic loading and if so, the appropriate measures to use to retrofit the existing bridges.

The two-volume report provides current information on the theory and techniques for seismic analysis of highway bridges, including background on basic structural dynamics, and demonstrates design details and installation specifications for retrofitting existing highway bridges to minimize earthquake damage.

Volume 1, **Earthquake and Structural Analysis,** includes a definition of terms, seismic risk maps, techniques and equations for analyzing existing bridges prone to earthquake loadings, and criteria for determining if a bridge needs retrofitting.

Volume 2, **Design Manual,** contains illustrations of various retrofit concepts and specific design procedures that can be applied to existing bridges.

Limited copies of the reports are available from the Implementation Division.



A Solar Heated Asphalt Storage Tank, Report No. FHWA-TS-78-207

by FHWA Implementation Division

This report evaluates the design and installation of a solar heated asphalt emulsion storage tank system in Oklahoma. The system, which is simple, inexpensive, and dependable, has been in operation for over a year and has more than fulfilled expectations. Such a solar heating system can be used throughout the year in a wide geographical area.

The cost of the solar installation is expected to be recovered in less than 2 years through reduced operating expenses. Cost advantages and uninterrupted heating capability are documented in the report by an analysis between the insulated solar tank and an adjacent uninsulated propane heated tank.

Special instrumentation to evaluate the performance of the solar installation measures the ambient temperature, the amount of solar radiation received on the collector face, the amount of heat circulating through the system at various points, and the temperature of the asphalt in the storage tank.

The report includes schematic diagrams, plans, piping, solar panels, controls, specifications, and performance data for the system. The concepts and data presented will aid those planning to design their own system.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 80-116890).

New Research in Progress

The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. Space limitation precludes publishing a complete list. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1I: Traffic Lane Delineation Systems for Adequate Visibility and Durability

Title: Design, Construction, and Evaluation of Installation Equipment for the Recessed Reflective Marker (RRM). (FCP No. 31I3113)
Objective: Modify the design of the experimental design vehicle intended to groove pavement and place recessed reflective markers automatically to eliminate overhanging subassemblies. Construct a prototype and prepare detailed engineering drawings. Verify the performance of the machine in normal field operations. Prepare and furnish an operators manual, maintenance and lubrication manual, and recommended spare parts list.
Performing Organization: B. A. Bodenheimer and Company, Stamford, Conn. 06905
Expected Completion Date: March 1982
Estimated Cost: $309,000 (FHWA Administrative Contract)

FCP Project 1Y: Traffic Management in Construction and Maintenance Zones

Title: Improved Safety Inspection Techniques for Highway Work Zones. (FCP No. 31Y1113)
Objective: Develop improved techniques for inspecting traffic controls in work zones through a review of existing procedures, examination of actual practices, and principles of positive guidance. Provide guidance on how and when to inspect traffic controls and how to recognize deficiencies.
Performing Organization: Transportation Systems Corporation, Vienna, Va. 22180
Expected Completion Date: November 1980
Estimated Cost: $149,000 (FHWA Administrative Contract)

FCP Category 2—Reduction of Traffic Congestion, and Improved Operational Efficiency

FCP Project 2K: Metropolitan Multimodal Traffic Management

Title: Development of Freeway Corridor Evaluation System—Passer IV. (FCP No. 42K2152)
Objective: Develop an integrated battery of analytical methods for evaluating urban freeway corridor transportation systems.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1981
Estimated Cost: $99,000 (HP&R)

FCP Category 3—Environmental Considerations in Highway Design, Location, Construction, and Operation

FCP Project 3F: Pollution Reduction and Environmental Enhancement

Title: Statistical Analysis of Ozone Data for Transportation/Air Quality Planning. (FCP No. 53F3492)
Objective: Develop methods for quantifying urban ozone levels from historical data and ozone monitoring to establish existing design values upon which ozone reduction strategies are based.
Performing Organization: Stanford Research Institute International, Menlo Park, Calif. 94025
Expected Completion Date: June 1981
Estimated Cost: $190,000 (NCHRP)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4C: Use of Waste as Material for Highways

Title: Data Bank for Recycled Bituminous Concrete Pavements. (FCP No. 34C4022)
Objective: Design and establish a comprehensive, cross-reference data bank containing information on construction projects using recycled bituminous concrete materials. Include data categories on location, soil types, drainage, materials, structural section, traffic loads, environmental factors, construction records, maintenance, and performance evaluation.
Performing Organization: Iowa State University, Ames, Iowa 50011
Expected Completion Date: March 1982
Estimated Cost: $100,000 (FHWA Administrative Contract)

Title: Evaluation of Asphalt Paving Recycling in Ohio: A Laboratory and Field Investigation. (FCP No. 44C4233)
Objective: Design recycled asphalt paving mixtures and evaluate asphalt modifiers for three Ohio paving projects. Evaluate the structural engineering properties of laboratory and field compacted test specimens. Predict and evaluate road performance and determine structural layer thickness equivalencies. Develop hot-mix asphalt recycling guidelines for testing, design, and construction.
Performing Organization: Ohio State University, Columbus, Ohio 43210
Funding Agency: Ohio Department of Transportation
Expected Completion Date: April 1982
Estimated Cost: $113,000 (HP&R)

FCP Project 4F: Develop More Significant and Rapid Test Procedures for Quality Assurance

Title: Asphalt Concrete Mixture Design and Specification. (FCP No. 44F2092)
Objective: Review Texas State Department of Highways and Public Transportation's asphalt concrete paving specifications. Attempt to improve density control, aggregate gradation, aggregate requirements, water susceptibility, mixture design methods, and job control and acceptance testing. Evaluate proposed specification changes on field construction projects.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1981
Estimated Cost: $93,000 (HP&R)

FCP Project 4G: Substitute and Improved Materials to Reduce Effects of Energy Problems on Highways

Title: Economic Skid Resistant Asphalt Surfaces. (FCP No. 44G2122)
Objective: Establish polish value requirements based on better correlations between polish values and field performance. Define polish value requirements based on traffic volumes in average daily traffic per lane.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1982
Estimated Cost: $160,000 (HP&R)

FCP Project 4J: Controlling Corrosion of Exposed Highway Structural Steel

Title: Low-Solvent Primer and Finish Coats for Use on Steel and Other Structures. (FCP No. 44J1133)
Objective: Evaluate and develop new paint systems, primers, and topcoats so that highway structures will be protected with the best possible coatings under the new air pollution control rules.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: September 1982
Estimated Cost: $227,000 (HP&R)

FCP Project 4K: Cost Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments

Title: Superplasticized Concretes for Bridge Decks and Highway Pavements. (FCP No. 34K2314)
Objective: Conduct a factorily designed experiment in the laboratory to determine the properties of superplasticized concretes with and without steel

fibers. Study sensitivities of the properties to water/cement aggregate content, admixture dosage, temperature, cement type, and other variables.
Performing Organization: South Dakota School of Mines and Technology, Rapid City, S. Dak. 57701
Expected Completion Date: August 1981
Estimated Cost: $128,000 (FHWA Administrative Contract)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5B: Tunneling Technology for Future Highways

Title: Field and Laboratory Evaluations of Camkometer. (FCP No. 25B2501)
Objective: Evaluate the camkometer for in situ testing of soil parameters and develop users manual for use by FHWA personnel.
Performing Organization: Federal Highway Administration, Washington, D.C. 20590
Expected Completion Date: November 1981
Estimated Cost: $99,000 (FHWA Staff Research)

Title: Evaluation and Design Utilization of Tunnel Instrumentation Data. (FCP No. 35B3122)
Objective: Develop more economical and safer guidelines for planning, designing, and constructing rock chambers and tunnels through collection and interpretation of existing construction records, instrumentation data, and geologic data. Compare with an analytical evaluation of movements and loads around the chambers.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Expected Completion Date: April 1982
Estimated Cost: $100,000 (FHWA Administrative Contract)

Title: Instrumentation and Evaluation of Slurry Wall Construction. (FCP No. 35B3142)
Objective: Collect and summarize instrumentation and geologic data obtained from Red Line Station. Observe Harvard Square construction procedures and evaluate data interpretation plans. Adopt finite element program to slurry wall analysis and implement approved plan for additional instrumentation and data analysis.
Performing Organization: Massachusetts Bay Transportation Authority, West Somerville, Mass. 02144
Expected Completion Date: September 1984
Estimated Cost: $356,000 (FHWA Administrative Contract)

FCP Project 5C: New Methodology for Flexible Pavement Design

Title: Flexible Pavement Data Base and Design. (FCP No. 45C3421)
Objective: Update the Texas flexible pavement data base on performance, distress, deflections, materials properties, traffic, and skid resistance. Convert the flexible pavement system (FPS) to layered elastic theory. Conduct tests to determine material properties of expansive soils.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: September 1983
Estimated Cost: $295,000 (HP&R)

FCP Project 5K: New Bridge Design Concepts

Title: Reevaluation of American Association of State Highway and Transportation Officials (AASHTO) Shear and Torsion Provisions for Reinforced and Prestressed Concrete. (FCP No 45K1122)
Objective: Make overall evaluation of current design procedures for shear and torsion, perform physical tests to facilitate evaluation, and develop practical design recommendations and proposed AASHTO specification requirements.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1982
Estimated Cost: $150,000 (HP&R)

Title: Fatigue Strength of Prestressed Concrete Girders. (FCP No. 45K1132)
Objective: Review current literature, conduct laboratory fatigue tests on full-size prestressed concrete girders, and develop recommended revisions to current design practice.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1982
Estimated Cost: $165,000 (HP&R)

FCP Project 5L: Safe Life Design for Bridges

Title: Improving the Reliability and Integrity of Electroslag Weldments in Bridge Structures. (FCP No. 35L1072)
Objective: Improve process control and develop microstructure controls, more reliable nondestructive testing, and special topics to improve the electroslag welding process. Fabricate weldments in accordance with a plan resulting from these studies, and evaluate the effect of process changes on mechanical properties.
Performing Organization: Battelle Memorial Institute, Richland, Wash. 99352
Expected Completion Date: January 1982
Estimated Cost: $332,000 (FHWA Administrative Contract)

Title: Optimization of Existing Nondestructive Bridge Inspection Devices for Routine Highway Bridge Inspection. (FCP No. 35L2012)
Objective: Evaluate existing acoustic crack detector/magnetic crack definer units and make recommended changes to improve performance. Develop breadboard units for laboratory and field tests of the improved units. Fabricate prototype units based on results.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: February 1981
Estimated Cost: $113,000 (FHWA Administrative Contract)

FCP Category 0—Other New Studies

Title: Desirable Asphalt Properties. (FCP No. 40M3603)
Objective: Develop test methods to define desirable asphalt properties and correlate properties with field performance. Investigate asphalt additives that are capable of improving marginal asphalts and determine the associated costs.
Performing Organization: Texas Transportation Institute, College Station, Tex. 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1984
Estimated Cost: $339,000 (HP&R)

Title: Fatigue of End-Bolted Cover Plates. (FCP No. 40S1712)
Objective: Examine the use of high-strength bolts in improving the fatigue life of the critical cover plate detail known to limit the fatigue strength of bridge girders with weldment. Determine experimentally the fatigue resistance of the end-bolted and retrofitted cover plate details. Evaluate test results using fracture mechanics concepts and statistical analysis techniques.
Performing Organization: University of Maryland, College Park, Md. 20742
Funding Agency: Maryland State Highway Administration
Expected Completion Date: December 1980
Estimated Cost: $93,000 (HP&R)

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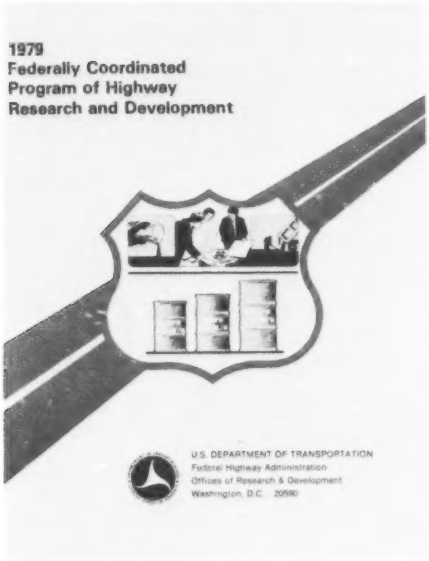
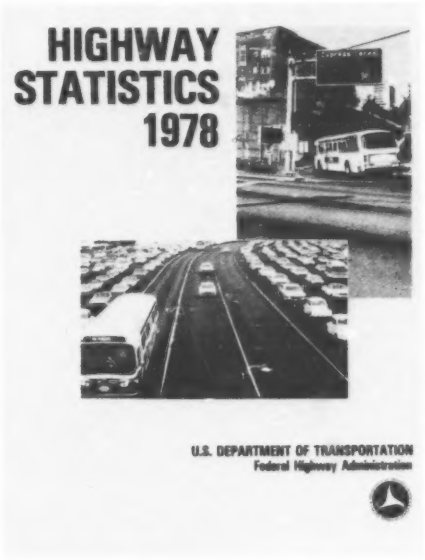
New Publications



The Offices of Research and Development (R&D), Federal Highway Administration (FHWA), have announced release of their fiscal year **1979 Annual Report on the Federally Coordinated Program (FCP) of Highway Research and Development.**

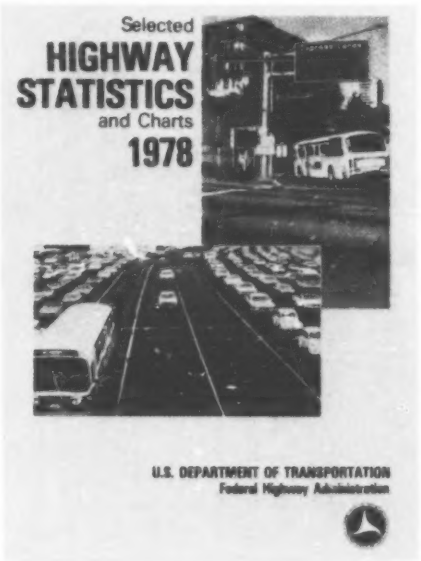
This year's 30-page report consists of two major parts. The first is organized into five major topics—safety research, traffic operations research, environmental research, structural research, and highway maintenance. This part is preceded by a brief statement on the background of the FCP. The second major part deals with organization, funding, and services of the Offices of R&D. The report is prefaced by a message from Federal Highway Administrator Karl S. Bowers.

While supplies last, individual copies of the report will be available free of charge to highway-related agencies and universities. Requests should be sent on agency or institution letterhead to the Federal Highway Administration, Engineering Services Division, HDV-14, Washington, D.C. 20590. Copies of the report are on sale for $2 by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00156-0).



Highway Statistics, 1978, the 34th publication in the annual series, presents statistical and analytical tables of general interest on motor fuel, motor vehicles, driver licensing, highway-user taxation, State and local highway financing, road and street mileage, and Federal aid for highways. Also reported are 1977 highway finance data for municipalities, counties, townships, and other units of local government. A listing of the data is given in the table of contents and a brief discussion is given in the text accompanying each section.

The publication may be purchased for $5.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00157-8). It is also available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Report No. FHWA-HP-HS-78).



Selected Highway Statistics and Charts, 1978, is a 32-page compilation of statistical highlights and charts prepared as a convenient summary supplement to various tables published in **Highway Statistics, 1978,** and previous publications in that annual series. Copies are available from the Highway Statistics Division, HHP-41, Federal Highway Administration, Washington, D.C. 20590.

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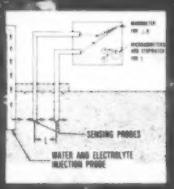
The Development of Improved Parking Signs



Determination of the In Situ Permeability of Bases and Subbases

The Seventh Annual Review of the FCP

The Frost Action Problem—An Overview of Research to Provide Solutions

Development and Testing of Advanced Control Strategies in the Urban Traffic Control System

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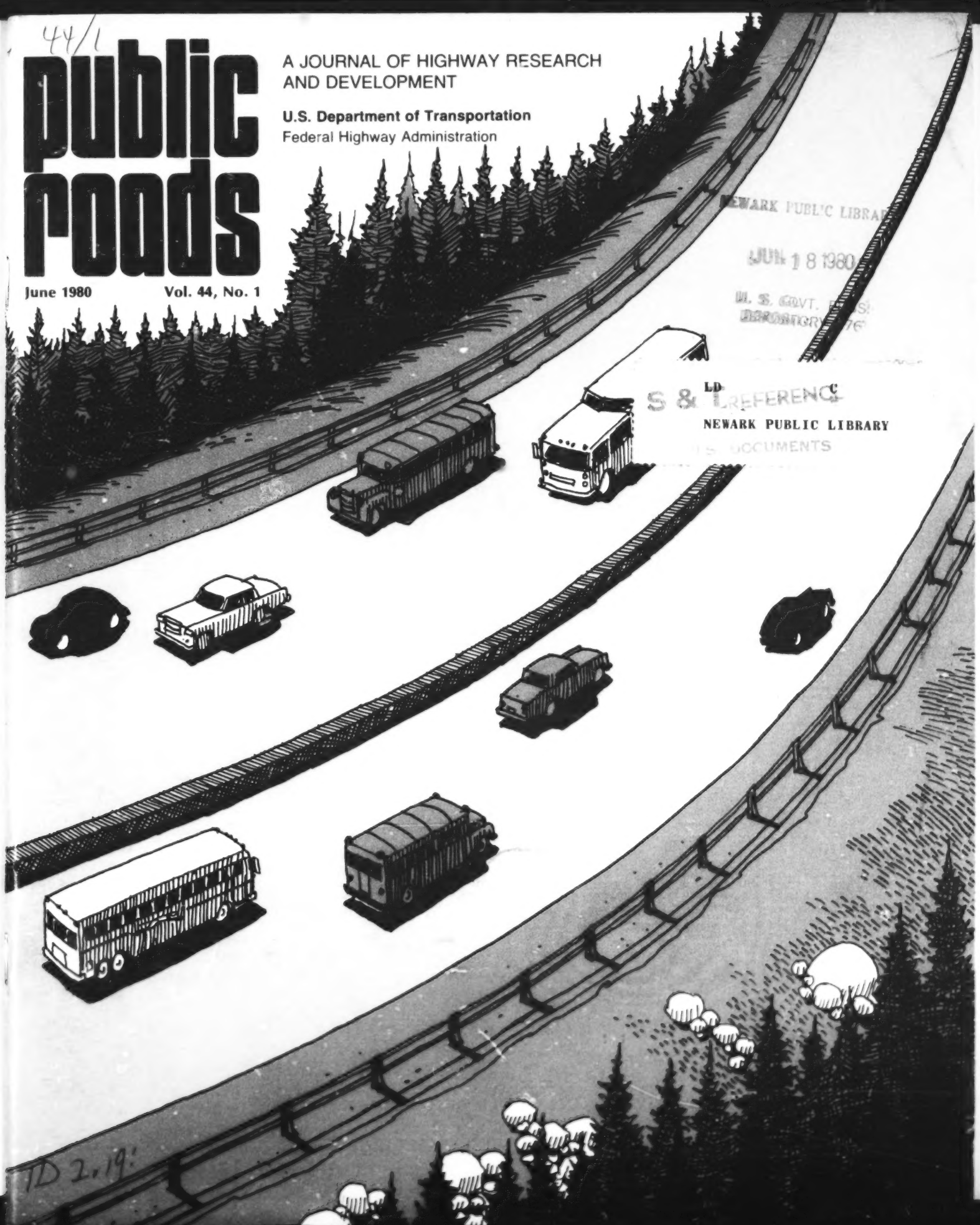
June 1980 **Vol. 44, No. 1**



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June 1980 Vol. 44, No. 1

COVER:
Roadside safety devices in relation to the downsizing of vehicles.

U.S. Department of Transportation
Neil Goldschmidt, *Secretary*

U.S. Department of Transportation
Federal Highway Administration
Washington, D.C. 20590

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Vehicle Downsizing and Roadside Safety Hardware

by
John G. Viner[1]

Introduction

Downsizing of domestic passenger vehicles is expected to be a major element in the strategy to meet the fuel economy mandates of the Corporate Average Fuel Economy (CAFE) requirements of the 1975 Energy Policy and Conservation Act. The expected increase in mini-sized vehicles makes it necessary to reexamine existing roadside safety hardware design and performance criteria, which generally are based on vehicles weighing 1.02 to 2.04 Mg (2,250 to 4,500 lb). An analysis of future vehicle downsizing needs suggests that vehicles as light as 0.77 to 0.81 Mg (1,700 to 1,800 lb) must be considered.

As vehicle weight decreases, the severity of crashes involving vehicles and pole-like structures increases. High-speed offcenter impacts of small vehicles with such structures may result in violent multiple rollovers. Thus it is important to reexamine the safety performance criteria for breakaway signs and luminaire supports, base bending signs, and other poles and supports.

Vehicle geometry, inertial characteristics, and crash properties of mini-sized vehicles may present special problems in the design of traffic railings. Small vehicles have snagged support posts and rolled over in traffic railing tests. Existing impact attenuators have been designed to accommodate vehicles weighing 1.02 to 2.04 Mg (2,250 to 4,500 lb). The performance criteria for these devices should be reexamined to consider the special problems of mini-sized vehicles.

This article reviews the evolution of vehicle size considerations in roadside safety hardware research. Data are discussed on the selection of initial mini-sized vehicles for Federal Highway Administration (FHWA) research. Results from ongoing research to identify safety problems in collisions of smaller vehicles and appropriate countermeasures are reviewed.

Ran-Off-the-Road Accidents

Ran-off-the-road accidents are a major highway safety problem and the development and use of roadside safety hardware has been an important strategy in reducing the severity of such accidents. The importance of this problem is illustrated in figure 1, which shows the distribution of fatal accidents in 1977 classified by first harmful

[1] This article is a condensation of an article in the *Proceedings of the Twenty-Third Conference, American Association for Automotive Medicine*, Louisville, Ky., Oct. 3–6, 1979.

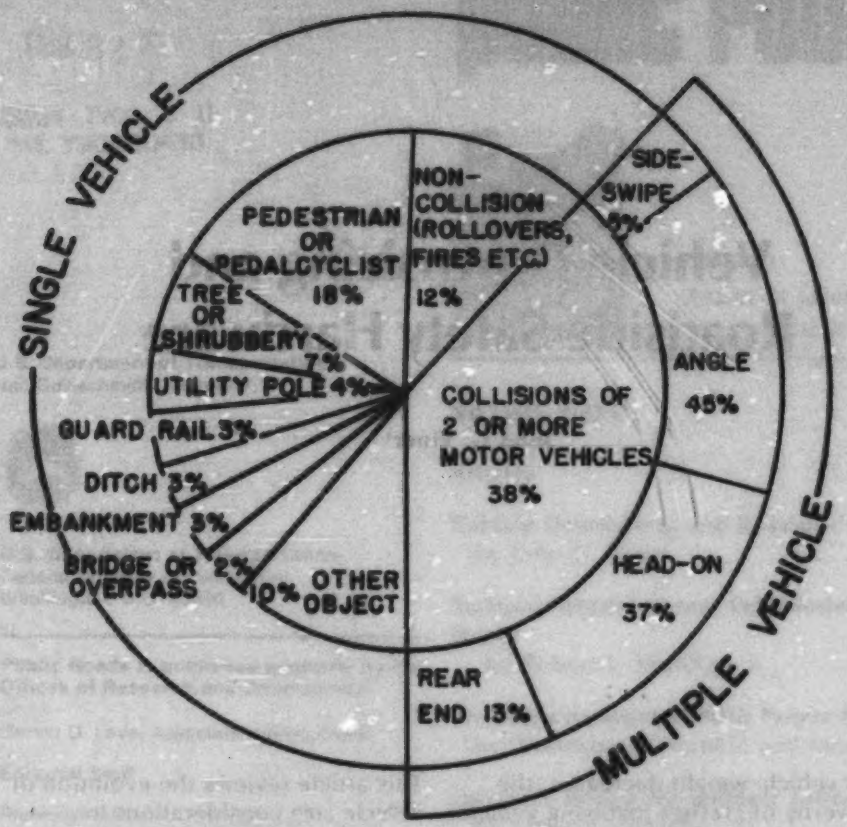


Figure 1.—Distribution of fatal accidents by first harmful event.

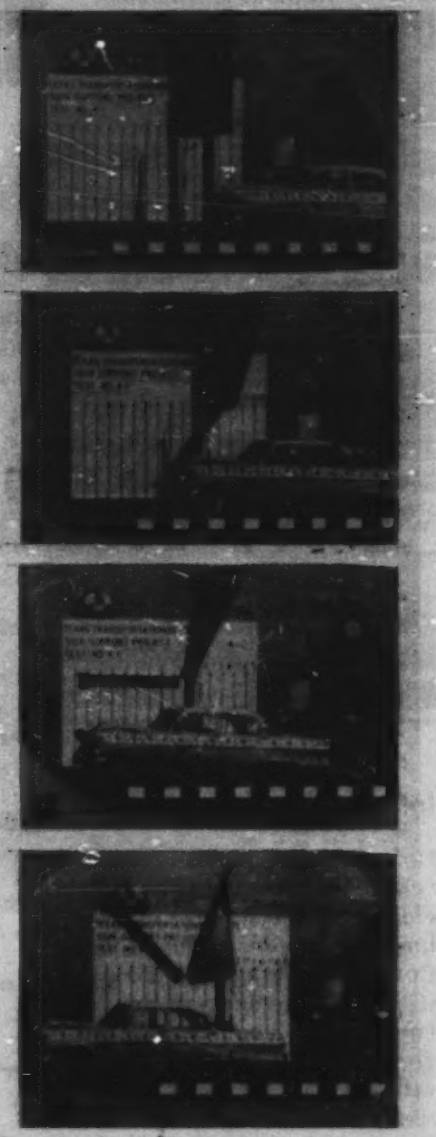


Figure 2.—Breakaway sign test—a 1.63 Mg (3,600 lb) vehicle traveling 64 km/h (40 mph).

event. (*1*)[1] Subtracting the pedestrian or pedalcyclist accidents from the single vehicle figures, it can be seen that in 44 percent of the 1977 fatal accidents a roadside feature was listed as the first harmful event. Because of the high speed involved in fatal accidents, one or more roadside features are involved in a significant fraction of the fatal accidents classified as multiple vehicle.

Impact Tests

Developmental work in the 1960's was essentially concerned with only passenger vehicles 1.59 to 1.81 Mg (3,500 to 4,000 lb). Often, only one impact condition was examined in full scale proof testing. Guardrail and median barrier crash tests were conducted with 1.81 Mg (4,000 lb) vehicles traveling 97 km/h (60 mph) with an impact angle of 25°, in accordance with a 1962 recommendation of the Highway Research Board Committee on Guardrails and Guideposts. (*2*) Only one vehicle—a mid-1950's standard-sized sedan weighing about 1.59 Mg (3,500 lb)—was used in the mid-1960's to research the development of the breakaway sign. (*3–5*) Figure 2 shows a 64 km/h (40 mph) test with a breakaway sign.

Actual vehicle crash tests on breakaway luminaire supports conducted in the late 1960's used

[1] Italic numbers in parentheses identify references on page 8.

vehicles weighing between 1.59 and 1.81 Mg (3,500 and 4,000 lb). Like the breakaway sign tests, all of these tests were frontal at the vehicle's centerline. In addition, the breakaway luminaire support crash tests were conducted at a single speed—64 km/h (40 mph). (6) Figures 3 and 4 show the results of such a test with a 12 m (40 ft) high luminaire support. In this test the change in vehicle velocity was 9.7 km/h (6.0 mph). The change in vehicle momentum was 4.40 kN-sec (980 lb-sec) and the maximum vehicle deformation was 430 mm (17 in). An analysis of this research predicted that severity of impacts with luminaire supports increased as vehicle weight and impact speed decreased.

Figure 4.—Vehicle after impact test with luminaire support shown in figure 3.

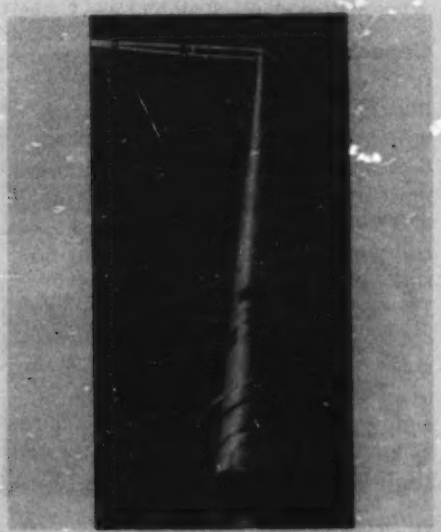


Figure 3.—A luminaire support used to test the impact with a 1.63 Mg (3,600 lb) vehicle traveling 64 km/h (40 mph).

In 1968 acceptance criteria were written for breakaway luminaire supports used on Federal-aid highways. The change in vehicle momentum was limited to 4.90 kN-sec (1,100 lb-sec) in such crash tests. (7) This limit corresponds to a 9.7 km/h (6.0 mph) change in velocity for a 1.81 Mg (4,000 lb) vehicle. Information available at the time suggested that serious head and chest injuries could occur to unbelted vehicle occupants when the vehicle underwent a sudden change in velocity—18 to 21 km/h (11 to 13 mph). (8–10) Thus, for the unbelted occupants this criterion was thought to provide a margin between expected vehicle behavior and serious head injury.

At the beginning of the research program on the development of impact attenuators, it was recognized that the effectiveness of impact attenuators greatly depended on the weight of the impacting vehicle. Impact attenuators were evaluated for vehicles weighing 0.91 to 2.04 Mg (2,000 to 4,500 lb) and traveling 97 km/h (60 mph) when the impacts occurred. (11) Figure 5 shows a vehicle weight distribution observed from traffic on Michigan freeways in 1968. (12, 13) More than 80 percent of the vehicles used in this initial impact attenuator developmental program weighed between 0.91 and 2.04 Mg (2,000 to 4,500 lb).

Figure 6 shows a 0.88 Mg (1,940 lb) vehicle traveling at 95 km/h (59 mph) in an impact attenuator test. (14) Figure 7 shows the vehicle after the test. The stopping distance in this test was 5.8 m (19 ft), which corresponds to an average deceleration of 59.8 m/s² (6.1 g). The maximum permitted deceleration in such a test is 117.6 m/s² (12 g). However, several attenuators produce decelerations of 58.8 to 78.4 m/s² (6.0 to 8.0 g) or even less under conditions specified in the test criteria, and the design and use of such attenuators have been encouraged. (15)

In 1974, procedures recommended for vehicle crash testing of traffic railings, impact attenuators, and breakaway or yielding supports were published to promote uniform testing practices for comparing the merits of two or more promising roadside safety devices. (16) Tests with 1.02 Mg (2,250 lb) and 2.04 Mg (4,500 lb) vehicles were specified for each device type, and a tolerance of ± 0.09 Mg (± 200 lb) was permitted

Figure 5.—Vehicle weight distribution from 1968 Michigan freeway data.

Figure 7.—Vehicle after test shown in figure 6.

Figure 6.—An impact attenuator test—an 0.88 Mg (1,940 lb) vehicle traveling 95 km/h (59 mph).

in the weight of the test vehicle. Figure 8 shows that approximately 85 percent of the passenger vehicles registered in the United States in 1974 weighed between 1.02 and 2.04 Mg (2,250 and 4,500 lb). (17) Domestic automobiles weighing about 1.02 Mg (2,250 lb) were a significant portion of new car sales in 1974.

Figure 9 shows a 1.02 Mg (2,250 lb) vehicle next to a shaped concrete median barrier. Figure 10 shows the difference between two barrier shapes—New Jersey (NJ) and General Motors (GM) profiles—widely used in the mid-1970's. (17) The results of tests with vehicles weighing 1.02 Mg (2,250 lb), traveling 97 km/h (60 mph) with an impact angle of 15° on the NJ and GM barrier shapes showed rollover with the GM shape but no rollover with the NJ shape. (16) These tests provide insight on collision severity under conditions more common than the previously used single test using a vehicle weighing 1.81 Mg (4,000 lb), traveling 97 km/h (60 mph), with an impact angle of 25°.

The frequency of rollovers of compact vehicles with the GM and NJ barrier shapes was found by real world accident analyses, vehicle crash testing, and computer simulations. Unlike compact vehicles, full-sized 2.04 Mg (4,500 lb) vehicles did not have a higher frequency of rollovers upon impact with the NJ barrier shape. Minor changes in barrier design produced important changes in the severity of impact of compact vehicles.

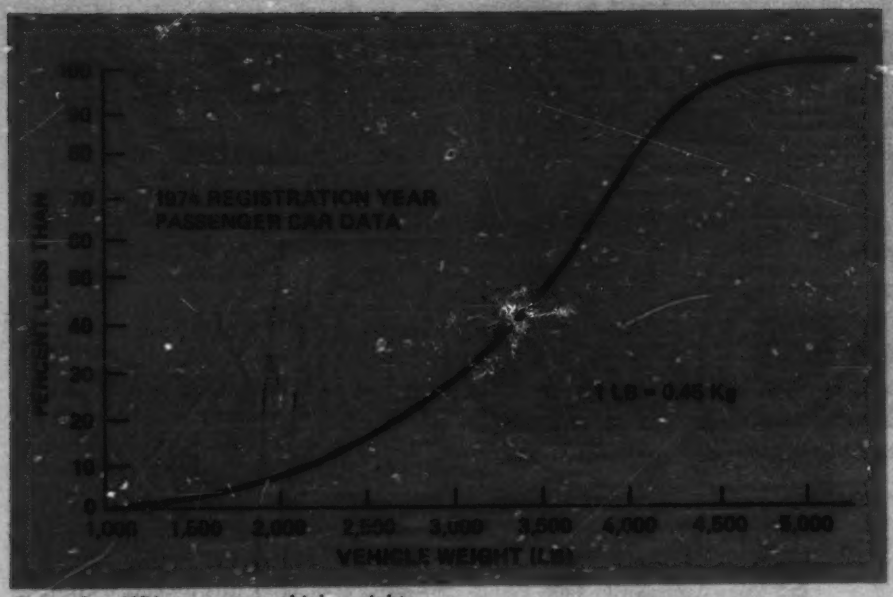


Figure 8.—1974 passenger vehicle weights.

Figure 11.—Intercity bus and guardrail accident.

Figure 12.—Straight truck and bridge rail accident.

Figure 9.—A 1.02 Mg (2,250 lb) vehicle and a shaped concrete median barrier.

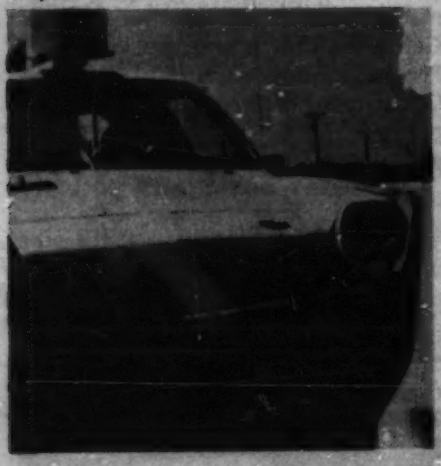
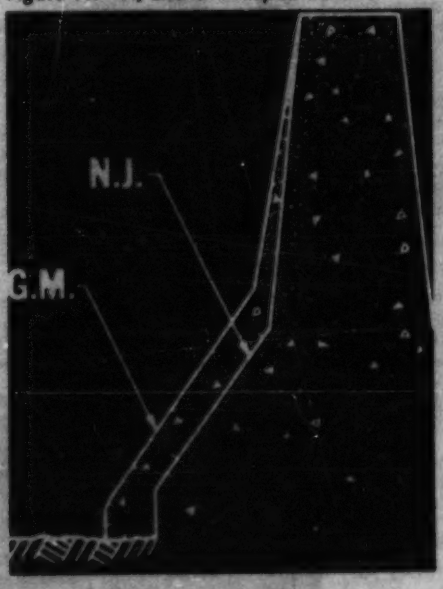


Figure 10.—NJ and GM shaped barriers.



The British reported impact tests of a mini vehicle weighing 0.76 Mg (1,670 lb) with NJ shaped concrete barriers. (*18*) Violent multiple rollover resulted in a 111 km/h (69 mph), 20° test with this vehicle.

Figures 11 and 12 demonstrate the need to develop traffic railings to retain heavy vehicles. FHWA research shows that it is feasible to develop traffic railings that can retain heavy vehicles without increasing collision severity for passenger car impacts. (*19*) Because these railings may be significantly more expensive than conventional railings, appropriate placement criteria are necessary to make the best use of available funds.

Developmental work on railings for heavy vehicles has focused on high occupancy vehicles—school buses, intercity buses, and tractor-trailers—with impact conditions as severe as 89 to 97 km/h (55 to 60 mph) at 15°. Figure 13 shows such a bridge rail and figure 14 shows the test results of an 18.14 Mg (40,000 lb) intercity bus traveling 89 km/h (55 mph), with an impact angle of 15°. (*20*) The bus was driven from the site after this test.

READEX

Figure 13.—18.14 Mg (40,000 lb) intercity bus impacting collapsing ring bridge rail.

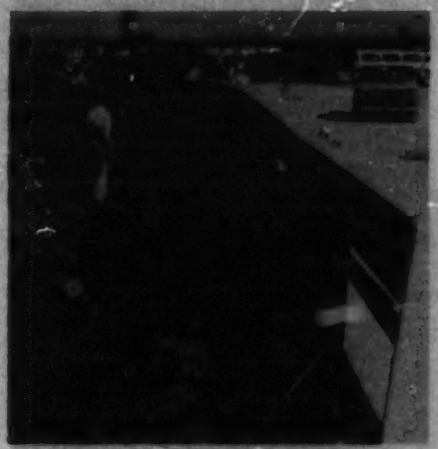


Figure 14.—Test results of 18.14 Mg (40,000 lb) bus traveling 89 km/h (55 mph), with an impact angle of 15°.

Figure 15.—Small sign support.

The updated recommended crash test procedures call for offcenter impacts, as stated above, with base bending signs to check for problems; however, the recommended vehicle weight of 1.02 to 2.04 Mg (2,250 to 4,500 lb) has not been changed. (23) An FHWA research study is planned to examine problems with smaller vehicles.

Meeting CAFE Requirements

The role of downsizing domestic passenger vehicles to meet the CAFE requirements has been predicted in a recent study. (24) Figure 17 shows the curb weight of domestic passenger vehicles manufactured in the United States in 1978 (actual weight) and 1986 (projected weight). A significant number of the domestic models expected to be produced in 1986 will weigh only 0.77 to 0.81 Mg (1,700 to 1,800 lb). A sales projection made by the Transportation Systems Center, Research and Special Programs Administration, U.S. Department of Transportation, predicts that 29 percent of 1985 passenger vehicle sales will be two

In a recent research program 1.02 Mg (2,250 lb) vehicles impacted small highway sign supports (fig. 15) head on at 97 km/h (60 mph) with the point of impact 381 mm (15 in) from the centerline. (21) In two of these tests vehicles overturned. This raised a concern about the increasing number of lighter vehicles with shorter wheel bases and narrower track widths that are on U.S. highways. In a limited followup effort, three tests were conducted with 0.88 Mg (1,940 lb) vehicles. A rollover occurred in one of these tests (fig. 16). (22)

Figure 16.—Test results of 0.88 Mg (1,940 lb) vehicle traveling 97 km/h (60 mph).

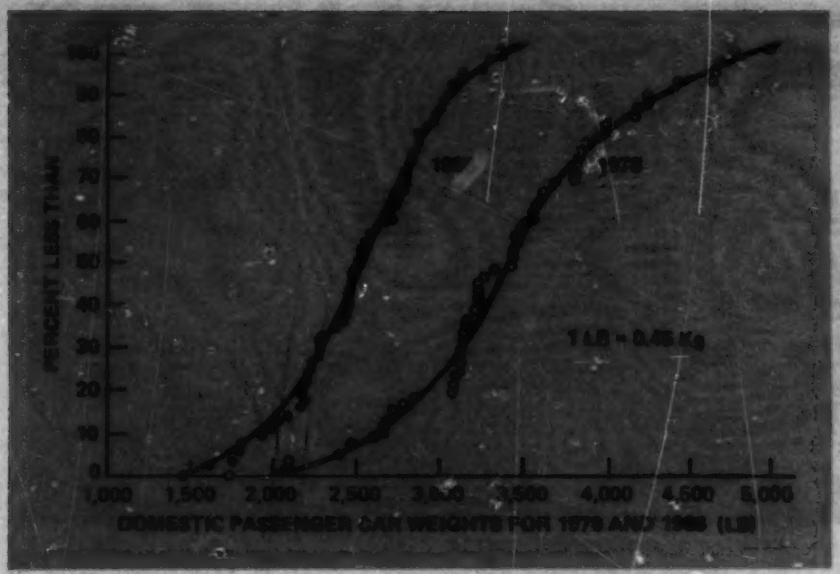


Figure 17.—Domestic passenger car weights for model years 1978 (actual) and 1986 (projected).

Figure 18.—Honda Civic and bridge rail impact test —0.81 Mg (1,800 lb) vehicle traveling 97 km/h (60 mph), with an impact angle of 15°.

seater (1 percent) or subcompact (28 percent) sizes.[3] Therefore, vehicles lighter than 1.02 Mg (2,250 lb) must be considered in the development and use of roadside safety hardware; FHWA research in this area will consider mini-sized vehicles. (25)

Selecting Mini-Sized Vehicles

Table 1 lists passenger vehicles weighing less than 1.02 Mg (2,250 lb) that were sold new in the United States in 1978. Values for a 1971 Vega Hatchback, a frequently used test vehicle in the 1.02 Mg (2,250 lb) class, are shown for comparison.

The Honda Civic has been selected as the initial mini-sized vehicle for FHWA research. This vehicle has front-wheel drive and a traverse mounted engine—a configuration expected to be used widely in future mini-sized vehicles.

Figures 18 and 19 show the test results of a Honda Civic traveling

Table 1.—1978 passenger vehicles in order of increasing base weight

Model and make	Wheel base	Tread		Individual model weight
		Front	Rear	
	(in)	*(in)*		*(lb)*
Honda Civic	86.6	51.2	50.4	1,664
Honda Civic CVCC	86.6	51.2	50.6	1,761
Renault LeCar	95.2	50.2	49.3	1,819
VW Rabbit	94.4	54.7	53.5	1,837
VW Scirocco	94.5	52.7	52.5	1,948
Fiat 128	96.5	51.3	51.8	1,950
Mazda GLC	91.1	51.0	51.6	1,965
Subaru DL	96.9	50.2	49.6	1,965
Chevette	94.3	51.2	51.2	1,991
Datsun F-10 Coupe	94.3	50.4	49.0	1,995
Subaru GF	96.9	50.2	49.6	2,015
Datsun B-210 Sedan	92.1	50.2	49.0	2,020
Honda Accord	93.7	55.1	54.7	2,030
Toyota Corolla	93.3	51.2	50.6	2,050
Audi Fox	96.7	52.7	52.5	2,070
VW Beetle	95.3	53.1	53.1	2,110
Omni	99.2	55.5	55.1	2,137
Horizon	99.2	55.5	55.1	2,137
Opel Coupe	94.3	51.4	51.4	2,150
VW Dasher	96.7	52.7	52.5	2,162
Arrow 160 Coupe	92.1	51.1	50.0	2,194
Opel Sedan	94.3	51.4	51.4	2,220
Datsun 510 Sedan	94.5	52.5	52.4	2,240
Vega Hatchback (1971)	97.0	55.0	54.2	2,250±

1 in=25.4 mm 1 lb=0.45 kg

[3] K. H. Schaffer and J. S. Yarmus, "Projection of U.S. Motor Vehicle Sales," Transportation Systems Center, Energy Demand Branch, Sept. 11, 1978. Unpublished memorandum.

89 km/h (55 mph) impacting a current bridge rail design at a 20° angle.[4] Much deceleration occurred when the vehicle snagged a bridge rail post in this test, and the right door window was broken by a dummy occupant. In a comparable test with a 1.02 Mg (2,250 lb) vehicle the vehicle redirection was much smoother. Wheel size and rail geometry are important factors in this problem. This study is still underway; recommendations to reduce such problems are planned.

Figure 19.—Vehicle after test shown in figure 18.

Summary

Existing roadside safety devices have played an important role in reducing the severity of ran-off-the-road accidents. The expected increase of mini-sized vehicles makes it necessary to reexamine highway safety hardware development and acceptance criteria. Important safety questions have been identified and work has begun to provide the needed answers.

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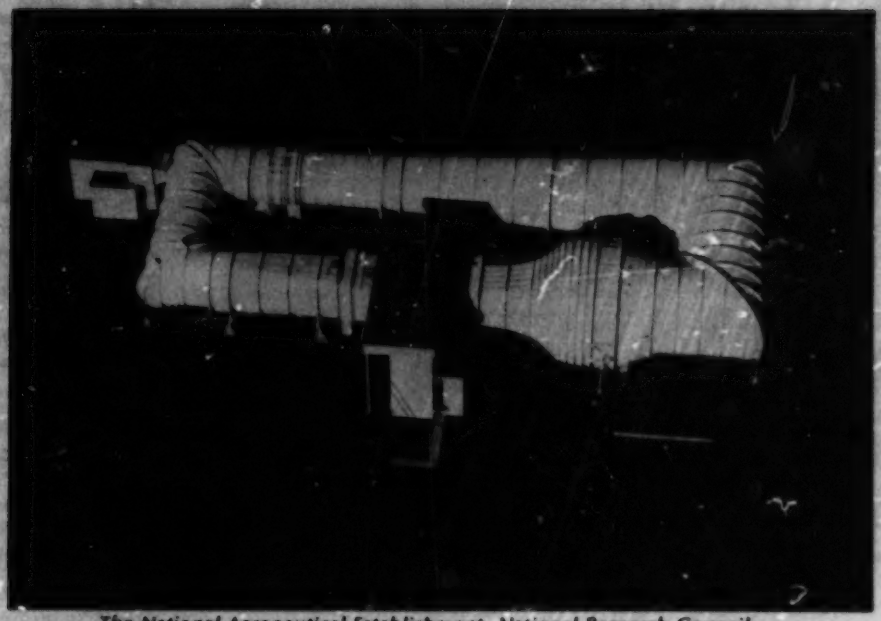
John G. Viner has been Chief of the Protective Systems Group, Structures and Applied Mechanics Division, Federal Highway Administration, since 1970. For the past 13 years he has been engaged in the physical aspects of highway safety research, having been involved in the development of roadside safety hardware. Prior to that he conducted structural vibration research for the Naval Ship Research and Development Center.



The National Aeronautical Establishment, National Research Council, 9 m by 9 m (30 ft by 30 ft) low speed wind tunnel in Ottawa, Canada.

Sectional Model Versus Full Model Wind Tunnel Testing of Bridge Road Decks

by
Robert L. Wardlaw

This article reviews fundamental engineering aspects of the wind tunnel modeling of bridge road decks. The advantages and disadvantages of current wind tunnel techniques for predicting the dynamic response of bridge road decks to wind are examined. The sectional model approach is shown to be reliable in predicting vortex shedding excitation and flutter. However, it may be excessively conservative because of the omission of wind turbulence in the simulation. Complete simulation of the Earth's surface wind is possible with the full model so that the effects of turbulence are fully accounted for. It is concluded that for major bridges, extensive aerodynamic investigations, including the more expensive full model, are justified.

Introduction

In a 1973 state-of-the-art review of bridge road deck aerodynamics, it was observed that the deck behavior could not be predicted satisfactorily by analytical means alone. (*1*)[1] Gaps in the knowledge of the effects of turbulence and terrain were evident. More extensive investigations of the motion of completed structures were necessary to obtain more information on structural damping levels and, more importantly, to provide a basis for the comparison of model and full-scale measurements.

Although these problems have not been entirely resolved, significant progress has been made in all areas through field and laboratory investigations in several countries including Australia, Canada, Japan, and the United States. Therefore, it is timely to discuss some of the implications of these advances on techniques and approaches to wind tunnel investigations. The advantages and disadvantages of sectional model versus full model testing should be reexamined; however, more research is needed before all questions can be answered.

The effect of the gustiness or turbulence in the Earth's surface wind on the bridge dynamic behavior is the major issue needing discussion. In addition, the

[1] Italic numbers in parentheses identify references on page 17.

adequacy of the traditional gust factor approach for estimating peak wind loads in turbulent wind must be challenged. The following questions arise:

- Does testing in smooth flow give overly conservative estimates of aerodynamic stability in natural wind?
- Can dynamic behavior in natural wind be predicted analytically from results obtained with sectional models in smooth flow?
- Can the Earth's surface wind be modeled adequately in the wind tunnel?
- Can the bridge be modeled precisely at reduced scale?
- Do the advantages of the sectional model approach—low cost, large scale, configuration flexibility, and short model preparation time—have to be sacrificed for the advantages of the full model—simulation of the natural wind, inclusion of three-dimensional aeroelastic effects, and inherent modeling of the bridge modal dynamics?

These and other questions on wind tunnel investigations of bridge behavior will be examined in this article. A compromise between the full model and sectional model—the taut-strip model—will also be discussed. (2)

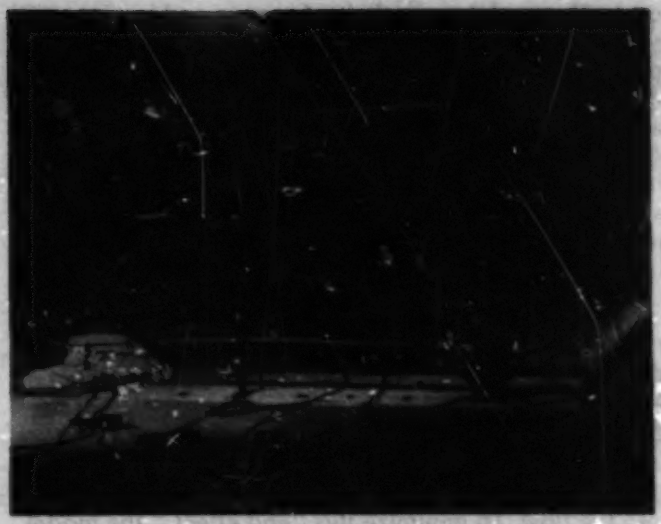


Figure 1.—The 1:110 scale full model of Lions' Gate Bridge in a 9 m by 9 m (30 ft by 30 ft) wind tunnel.

Figure 2.—Closeup view of the model shown in figure 1.

Wind Tunnel Testing Techniques

The full model

The full model, a reduced scale, geometric facsimile of the entire prototype bridge, includes all structural elements—the towers, the suspension cables, the road deck, and the road deck hangers. For dynamic studies, the mass, the mass distribution, and the elastic characteristics of the prototype must also be modeled according to established scaling principles. The scale ratio for a long-span bridge may be very small; for example, the model of a bridge 1.0 km (3,280 ft) long would have to be constructed at a scale ratio of about 1:400 to be tested in a wind tunnel test section 2.5 m (8.2 ft) wide. Clearly, large reductions in size present difficulties in model design and construction. Figures 1 and 2 show a 1:110 scale full model in a 9 m by 9 m (30 ft by 30 ft) wind tunnel. The model is a widened version of the Lions' Gate Bridge that crosses the Burrard Inlet in Vancouver, British Columbia, Canada. Aerodynamic "spires" at the entrance to the test section (fig. 1) generate the shear and turbulence properties of the Earth's surface wind. The depth of the wind layer, which depends on the roughness of the terrain at the bridge site, will be about 300 to 600 m (985 to 1,970 ft); consequently, at a scale of 1:110, the wind layer will be between 2.7 and 5.5 m (9.0 and 18.0 ft) deep in the wind tunnel.

The sectional model

The term sectional model is derived from the aeronautical engineering practice of measuring the two-dimensional or sectional properties of aerofoils in wind tunnels by using constant section models that span the test section.

Rather than model the entire bridge, a model representing a short, midspan section of the deck can be constructed to study the aerodynamics of the bridge road

deck. The model spans the test section and is supported rigidly at the walls for force measurements or mounted on pairs of springs for dynamic measurements. For dynamic measurements, the mass, the mass distribution, and the elastic properties must be modeled according to scaling criteria, as is required with the full model. The bending mode natural frequency is controlled by the spring stiffness, and the ratio of the bending to torsional mode frequencies is controlled by the spacing between the two component springs of the pair located at each end of the model.

Figure 3 shows a 1:30 scale sectional model of Long's Creek Bridge, a cable-stayed orthotropic girder bridge on the Trans-Canada Highway in New Brunswick, Canada. The model was tested in a vertical wind tunnel 4.6 m (15.0 ft) in diameter. The junction between the ends of the model and the wall introduces three-dimensionality into the flow. To minimize this "end effect," the model should be as slender as is practical; a span-to-chord ratio of 4:1 would be acceptable. For testing in a test section 2.5 m (8.2 ft) wide, a model chord of 0.6 m (2.0 ft) would be suitable; for a bridge road deck 30 m (98 ft) wide, this would give a scale ratio of 1:50.

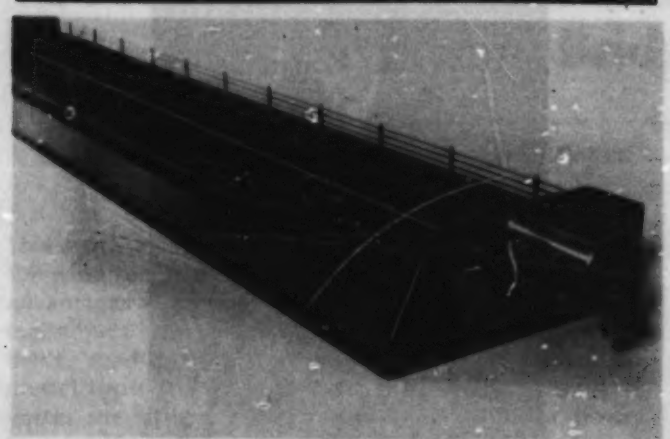


Figure 3.—The 1:30 scale sectional model of Long's Creek Bridge (original road deck configuration).

Because the sectional model is simpler and has a larger scale than the full model, it is less expensive to construct. Also, the sectional model can be built more quickly and can be modified quickly for examining corrective configuration changes. Although the air flow can be made turbulent (for example, by using coarse upstream turbulence grids), it is not fundamentally possible to simulate all of the natural wind properties with the sectional model, particularly the large size of the gusts. Research to develop large longitudinal scale, two-dimensional turbulence will be particularly useful in sectional model studies. (3)

Although road deck response to natural wind turbulence cannot be determined directly, it may be possible to analytically predict the response from sectional aerodynamic measurements made in smooth flow. (4) If successful, this approach would produce an ideal method for predicting bridge behavior in natural wind. However, problems still remain. One question that must be resolved is whether the aircraft aerodynamicist's classical strip theory applies to bridge road decks. This theory divides an airfoil into thin chordwise strips that are assumed to behave independently so that the overall airfoil performance can be calculated by spanwise integration.

It is common for slender elastic structures to be aerodynamically unstable because of their cross-sectional geometry—energy extracted from the airstream causes oscillatory motion of the structure. As will be discussed later, different aeroelastic mechanisms may account for the instability. In some cases the phenomenon is amplitude limited, but even though excessive stress levels may not result, the user will consider the bridge road deck unacceptable. (5) Other mechanisms can cause catastrophic motion levels in a few motion cycles.

It has been conventional practice to use the spring-mounted sectional model in smooth flow to assess road deck aerodynamic stability. For amplitude-limited excitations, amplitudes must not exceed a prescribed criterion of acceptability. (5) For catastrophic instabilities, motion must occur only at windspeeds greater than those expected at the site. Field and wind tunnel experience indicates that the sectional model conservatively predicts instabilities on prototype bridges. No unacceptable aerodynamic instability of prototype bridges has been observed when the road decks were wind tunnel tested in smooth flow before construction. The Golden Gate Bridge, the original Tacoma Narrows Bridge in Washington State, and the Long's Creek Bridge were not tested before construction and were aerodynamically unstable. (6-8) Subsequent wind tunnel studies in smooth flow confirmed the instabilities and demonstrated that the problems could have been avoided if wind tunnel testing had been done in advance.

The taut-strip model

With the taut-strip model, the road deck is attached to two parallel taut wires suspended across the wind tunnel test section. (2) The vertical and horizontal bending

mode natural frequencies are controlled by the wire tension, and the ratio of the bending mode to torsional mode natural frequencies is controlled by the distance between the wires.

Advantages of the taut-strip model over the sectional model are that natural wind turbulence can be appropriately simulated and the three-dimensionality of the model deformations are inherent. In addition, the taut-strip model is simpler in concept than the full model.

The taut-strip model scale is constrained not so much by the size of the prototype bridge as by how deep a wind layer, and thus how large a turbulence scale, can be accommodated in the wind tunnel. Wind layer depths may be about one-half the height of the wind tunnel test section. Therefore, in a test section 2.4 m (7.9 ft) high, a wind layer 1.2 m (3.9 ft) deep could be developed, representing a full-scale layer 300 m (985 ft) deep at a scale ratio of 1:250. If the model were installed in a test section 2.4 m (7.9 ft) wide, 600 m (1,970 ft) of bridge road deck could be represented. Although the taut-strip model may offer some advantage in scale over the full model, the scale will still be much smaller than that of the sectional model.

Model Scaling Considerations

Model scale observations can only be extrapolated with confidence to prototype scale if sound scaling principles are used in the design of the model and the experiment. This insures that the relative magnitudes of the forces involved in bridge dynamics—gravitational, inertial, aerodynamic, elastic, and structural damping—are the same for the model and the prototype and that the motion amplitudes are in the same proportion as the geometric scale ratio.

Dynamic scaling

The model scale value of the wind velocity must be lower than the full scale value by the square root of the geometric scale ratio for a full model of a long-span bridge to insure that gravitational forces are correctly proportional to aerodynamic and other bridge dynamic forces. Thus, for a scale ratio of 1:100, a full-scale windspeed of 100 km/h (62 mph) would be represented by a model scale windspeed of 10 km/h (6.2 mph) in the wind tunnel.

It follows that the elastic properties have to be adjusted so that the stiffnesses are lower than they would be for a replica model that is made from the same materials as the actual bridge. For some structures, this can be done by using materials that have lower values of the elastic modulus, E, and by reducing material thickness. The resulting deficiency in mass can be corrected by adding distributed nonstructural mass. For open truss designs, however, the elastically scaled values of component thickness are usually impractically small and some other way of constructing the model has to be devised. A structural spine, on which are mounted nonstructural modules that give the correct aerodynamic shape, can be designed for the road deck.

Aerodynamic scaling

Aerodynamic scale effects can cause problems when extrapolating model results to full scale. The wind flow at model scale may not be "similar" to flow at full scale. For example, at high windspeeds, the flow changes over a large diameter circular cylinder. The Reynolds Number, the aerodynamic similarity parameter, is a measure of aerodynamic scale. The Reynolds Number is as follows:

$$Re = \rho U b / \mu$$

Where,

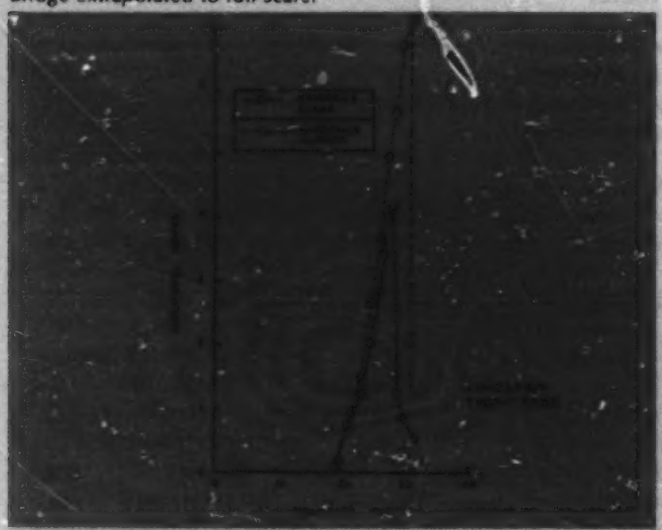
ρ = Air density.
U = Windspeed.
b = Cylinder diameter or bridge deck width.
μ = Air viscosity.

At large values of the Reynolds Number, the drag coefficient ($C_D = D/\frac{1}{2}\rho U^2 b$, where D = aerodynamic drag/unit length) for a circular cylinder abruptly drops in value. This is because the cylinder wake becomes narrower—a consequence of a rearward shift on the cylinder surface of the "separation points" where the smooth flow on the forward part of the cylinder detaches from the surface of the cylinder. The separation point is fixed at the sharp upstream corner of rectangular cylinders; as a consequence, the Reynolds Number dependency disappears. This is not the case for rectangles with rounded corners. At a very small scale, it is difficult to reproduce precisely the prototype geometric shape, particularly the small corner radii of structural components.

It is difficult to answer the question of how large the model scale should be to avoid Reynolds Number problems. Model-prototype comparisons and wind tunnel investigations over a range of scales indicate that Reynolds Number scaling is not necessary. Wind tunnel sectional model measurements of the Long's Creek Bridge (fig. 4) at a 1:30 scale are extrapolated to prototype scale in figure 5. The peak amplitude of 110 mm (4.3 in) near 44 km/h (27 mph) agrees with

Figure 4.—The modified Long's Creek Bridge.

Figure 5.—Sectional model response of the unmodified Long's Creek Bridge extrapolated to full scale.



several observations at the bridge site. The narrow velocity range of excitation is the same for the model and the prototype. Winds normal to the bridge at velocities above the critical range—56 km/h (35 mph)—were measured at the site and no bridge motion was observed. Amplitudes as high as 200 mm (7.9 in) were also reported. These larger responses were probably because of snow blocking the handrails. In the wind tunnel a peak amplitude of 180 mm (7.1 in) was recorded with blocked handrails.

Effects of Turbulence

The Earth's surface wind layer

The planetary winds are retarded near the Earth's surface by flow resistance from elements on the ground and by fluid friction associated with the air viscosity. As a result of this shearing action, the velocity varies from zero at the surface—the fluid dynamicist's no-slip condition—to gradient wind at about 300 to 600 m (985 to 1,970 ft) above the surface as shown in figure 6.

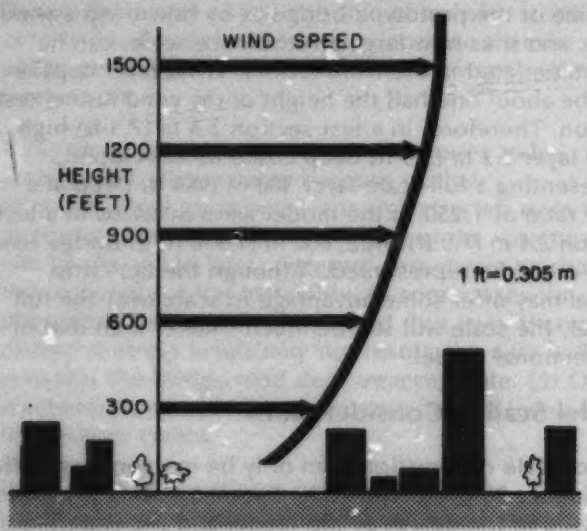


Figure 6.—The variation of mean wind velocity with height.

The shearing action also causes mechanical agitation of the flow, or turbulence. To an observer fixed on the ground the turbulence manifests itself as gustiness with continual and sometimes abrupt changes of direction and magnitude. A measure of the magnitude of the wind's fluctuating component is its root-mean-square (RMS) value, which is known as the turbulence intensity. At the height of the gradient wind, the turbulence intensity is nearly 0. However, it increases toward the surface, and the longitudinal or streamwise component can be as high as 30 to 40 percent of the local mean windspeed near the ground. Peak excursions can be several times the RMS value.

The wind turbulence is characterized by a nearly random distribution of the physical size of the disturbances. In the longitudinal direction the length can vary from near zero to hundreds of metres (feet). The lateral dimensions of the disturbances are somewhat smaller. The random

fluctuations in wind velocity, seen by a fixed observer, represent wind energy distributed over a range of frequencies.

Modeling the wind layer

There are several methods for simulating wind layer properties in the wind tunnel. A well-established approach used in special-purpose wind tunnels with long working sections is to develop the layer "naturally" by having the wind blow over a long fetch of surface elements. A second method, suitable for aeronautical wind tunnels with shorter working sections is to install spires at the entrance to the test section as shown in the background of figure 1. Both of these techniques make it possible to develop wind layer depths that approach one-half the height of the test section.

Testing in turbulent wind

The effects of turbulence on bridge behavior depend on the scale of the turbulence relative to the size of the bridge and the intensity and frequency spectrum of the turbulence. For example, if the lateral scale of the turbulence were large enough that the velocity at any one instant was constant along most of the span, turbulence would play a different role than if the scale were much smaller and the velocity varied considerably along the span. Similarly, if the natural frequencies of the bridge were in a high energy part of the spectrum, the response to turbulence would be greater than if the frequencies were in a low energy part. It is important to simulate all of the properties of turbulence. This cannot be done with sectional models because large scale turbulence components are absent. Therefore, the full model or taut-strip model must be used to fully study the effects of turbulence.

Buffeting response

Because there is significant turbulent energy near bridge natural frequencies, the dynamic response becomes important at high windspeeds for lightweight, low damping, modern suspended bridges, and the gust factor approach to estimating loads may not be satisfactory. An alternative is to use full models in simulated wind to estimate the stress levels caused by the dynamic response from motion. (9) As experience with full models increases, the goal of predicting dynamic response from sectional model data should be pursued vigorously.

Road deck flutter

Flutter is an oscillatory instability of the road deck that occurs above a critical windspeed. It is characterized by rapid buildup of amplitude; catastrophic levels may be reached in a few cycles of motion. At higher windspeeds the buildup rates normally increase. Torsion is usually the dominant vibration mode, but flutter is a consequence of aerodynamic coupling between vertical bending and torsion, although single-degree-of-freedom torsional flutter may occur. Unless the shape of the bridge is aerodynamically stable, it is essential that critical flutter speeds be well above windspeeds expected at the site.

Flutter stability can be readily assessed using sectional models. These assessments seem to be a conservative approach to design against flutter because the stability in turbulent flow will be greater than in the uniform flow of the sectional model test. There is evidence that the sectional model method can be too conservative. The Halifax Harbour Bridge (2) and the Lions' Gate Bridge (9, 10) are open truss bridges, and for both bridges, the flutter instability observed with sectional models was not observed for the full models in the turbulent flow, even for velocities well above the sectional model critical speeds. Torsional response of the Lions' Gate Bridge is shown in figure 7.

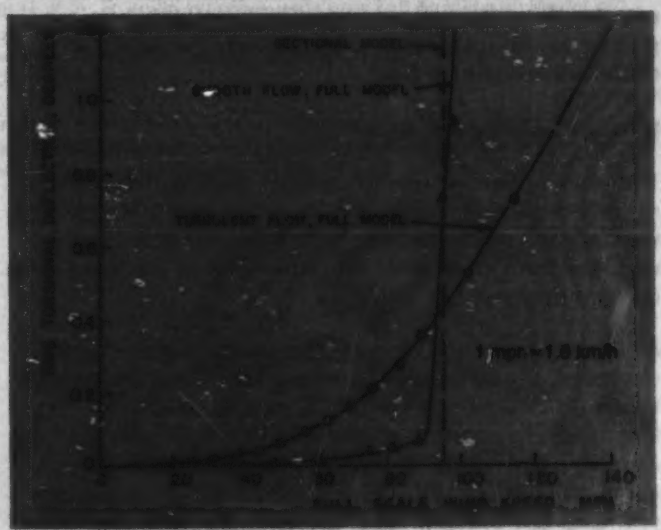


Figure 7.—The Lions' Gate Bridge full model torsional response at the 1/10th span position in smooth and turbulent flow. (10, 11)

Evidence that the sectional model is conservative is less compelling for the plate girder and box section road deck structures than for the open truss design. With the taut-strip model of the H-section road deck of the original Tacoma Narrows Bridge, turbulence inhibited

Figure 8.—The vortex wake behind a circular cylinder.

noncatastrophic, possibly vortex-induced, motion at subcritical windspeeds but only marginally postponed catastrophic torsional motion. (2) The effects became greater as the turbulence intensity was increased to represent rougher terrain.

Vortex shedding response

The wake structure behind slender, bluff structures is an orderly sequence of vortexes, as illustrated in figure 8 by visualization of the flow behind a circular cylinder. The vortexes form alternately on either side of the wake with marked periodicity.

As the windspeed is increased over a slender, elastic structure, the frequency of vortex formation increases. When this frequency matches a structural natural frequency, large crosswind vibration amplitudes develop. The critical windspeed range often will be at common windspeeds. The vertical bending oscillations of the Long's Creek Bridge, shown in figure 5, are caused by the periodic wake vortexes. (8) The finite, nondestructive amplitudes and the narrow velocity range over which the motion occurs are characteristic of vortex shedding response. Torsional modes are equally susceptible to this form of excitation. Although in the short term the amplitudes do not damage the bridge, they can be large enough to make the bridge unacceptable to the user and may result in long term damage.

Sectional models are suitable for studying vortex shedding response because easily made configuration changes are a helpful feature for developmental or remedial investigations. Experience with the Long's Creek Bridge demonstrates that the sectional model can reliably predict full-scale vortex shedding behavior for plate girder decks. Whether the sectional model is overly conservative for more streamlined box sections, for truss bridges, or for higher turbulence levels has yet to be established.

Conclusions

Although advantages and disadvantages of the full, sectional, and taut-strip model approaches have been discussed, it is not possible with the present state of the art to favor one model approach. The investigative procedure or combination of procedures to be used will depend on a variety of considerations—the size of the bridge, the road deck configuration, and the leadtime available.

As previously stated, advantages of the sectional model are low cost, large scale, configuration flexibility, and short model preparation time. The model also provides data for prediction procedures now under development. (4)

The advantages of the full model are that the Earth's surface wind can be completely simulated so that the

turbulence response is obtained directly and the effects of turbulence on flutter and vortex shedding are intrinsic. Also, all three-dimensional aeroelastic effects are modeled implicitly.

The taut-strip model can also be tested in simulated natural wind but it is simpler and less costly than the full model. Within the limits of this simplified model, some of the three-dimensional aeroelastic effects are included. To a certain extent it shares the low cost and short model preparation time of the sectional model. Analytical procedures for predicting prototype behavior from taut-strip measurements have been described in the literature. (*2*)

For a large bridge, extensive aerodynamic investigations are justified and can include a full model wind tunnel investigation. It can be advantageous to precede the full model investigation with sectional investigations so that stable aerodynamic deck shapes, free from vortex shedding excitation, are developed quickly and cheaply. This will also permit the assessment of Reynolds Number effects for the small scale full model. If the full model is difficult to justify for smaller bridges, the sectional model or the sectional model plus taut-strip model can be used.

In choosing the testing procedure it should be remembered that the cost of the aerodynamic investigation is a fraction of 1 percent of the cost of the full-scale structure. However, because of the importance of the aerodynamic behavior to the final design, the selection of the investigative technique should not be unduly restricted by cost considerations.

Although our knowledge of the dynamic behavior of bridge road decks and of investigative procedures has improved, there are still gaps to fill, particularly concerning turbulence, three-dimensional effects, and analytical prediction procedures. Future research should emphasize model to full-scale comparisons to further validate investigative procedures.

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[3] Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

Robert L. Wardlaw has been with the National Research Council of Canada since 1949. Until 1955 he worked in the Low Temperature Laboratory and since then has been in the Low Speed Aerodynamics Laboratory where he is Deputy Laboratory Head. Mr. Wardlaw supervises the laboratory program in wind engineering, which focuses on wind tunnel investigations of the aerodynamics of buildings, towers, chimneys, bridges, power cables, and surface vehicles and other nonaeronautical problems.

Problems Associated With Power Failures at Signalized Intersections

by
Woodrow W. Rankin
and Merton J. Rosenbaum, Jr.

Some traffic engineers have expressed concern about the potential adverse safety effects of power failures at signalized intersections. This article discusses the results of a workshop and survey that indicate that power failures at signalized intersections, particularly areawide failures, are not a significant safety problem.

Introduction

Periodically, the Federal Highway Administration (FHWA) Office of Highway Safety asks State and local jurisdictions to help identify critical subjects for highway safety research. In 1978 traffic engineers from Erie County, N.Y., San Juan, Puerto Rico, and the Florida Department of Transportation submitted problem statements requesting that power failures at signalized intersections be studied. They specifically requested the following:

- Develop methods for interim control of traffic signals when power fails during peak traffic periods.
- Evaluate potential safety hazards associated with power failure at signalized intersections.
- Develop criteria and warrants for installing power "failsafe" devices at signalized intersections.

Because the type and size of power failures at signalized intersections were not well defined, the FHWA Office of Research sponsored a small study to determine the potential seriousness of power failures at signalized intersections. The results of this study, which was performed by the Highway Users Federation for Safety and Mobility (HUFSAM), are summarized in this article as are complementing data from an independent national survey conducted by Herman Kimmel and Associates (HKA). The data collected by HKA and discussed in this article were analyzed by FHWA personnel.

The study techniques used by HUFSAM and the survey techniques used by HKA are briefly discussed. Also discussed are areawide and localized power disruption, the relationship of power failures to traffic accidents, and procedures used by operating agencies when signal outages occur.

Study and Survey Methodology

HUFSAM reviewed published and unpublished literature and conducted a workshop with traffic engineers from Dade County, Fla., Indianapolis, Ind., New York, N.Y., Tulsa, Okla., and the Illinois Department of Transportation to study power failures at signalized intersections. Additional information was obtained from traffic engineers

Related to the problem of signalization, we'd like to obtain information on the extent and frequency of *Power Failures at Signalized Intersections.*

1. Do you consider power failures at signalized intersections a major problem in your jurisdiction? (Check one):
   MAJOR ☐ MINOR ☐ NO PROBLEM ☐
2. How would you rank the major causes of power failure in your area?
   1. _____ 2. _____ 3. _____ 4. _____
3. What measures are taken to solve the problem?
   NONE ☐ BACKUP SYSTEM ☐ OTHER _____
4. Could you estimate the number of power failures at signalized intersections per year in your jurisdiction?
   Less than 50 ☐ 50-100 ☐ 100-200 ☐ 200+ ☐
5. What legal issues exist related to this problem? _____
   _____
6. Please estimate the number of signalized intersections in your state by the following categories:
   RURAL _____ SUBURBAN _____ URBAN _____ TOTAL _____
7. What are some of the options available to solve the problem of power failure at signalized intersections?
   _____
   _____
8. Is there evidence of an increase in traffic accidents when signals are out due to power failure? Rate severity of accidents. LOW MEDIUM HIGH YES ☐ NO ☐
9. What issues do you believe should be investigated related to power failures at signalized intersections?
   _____
   _____
10. Would you be willing to serve on a panel to answer further questions on power failures at signalized intersections?
    YES ☐ NO ☐

Figure 1.—HKA questionnaire on power failures at signalized intersections.

responsible for the operation and maintenance of signal systems in Baltimore, Md., Milwaukee, Wis., San Diego, Calif., Erie County, N.Y., King County, Wash., and Montgomery County, Md. The Maryland Department of Transportation and some manufacturers and suppliers of signal equipment were also contacted by HUFSAM during the study. Each traffic engineer who furnished information is responsible for the operation and maintenance of more than 300 signalized intersections.

In HKA's survey, a questionnaire (fig. 1) was sent to over 2,500 traffic engineers located in or associated with local jurisdictions and State agencies. A range of jurisdictions responded to the questionnaire including Coronado, Calif., with 10 traffic signals and the California Department of Transportation with 130,000 traffic signals. Figure 2 shows the geographical distribution of the responses.

Power Failures

Causes of signal outages can be divided into three categories: Areawide irregularities in power supply, localized irregularities in power supply, and signal equipment failure not related to power supply. Because this article is concerned with power failure related outages, only the first two categories will be addressed.

Areawide power disruptions

Areawide power disruptions include bulk power interruptions (blackouts), load reductions (brownouts), and other emergency situations. In the first 6 months of 1978, 20 blackouts were reported in the United States compared to 49 blackouts reported in 1977. (*1-4*)[1] These power interruptions lasted from 5 minutes to 6 days for various large groups of customers. Such large outages were caused by natural events—tornadoes, hurricanes, heavy flooding, severe ice storms, earthquakes, and other disasters—as

[1] Italic numbers in parentheses identify references on page 25.

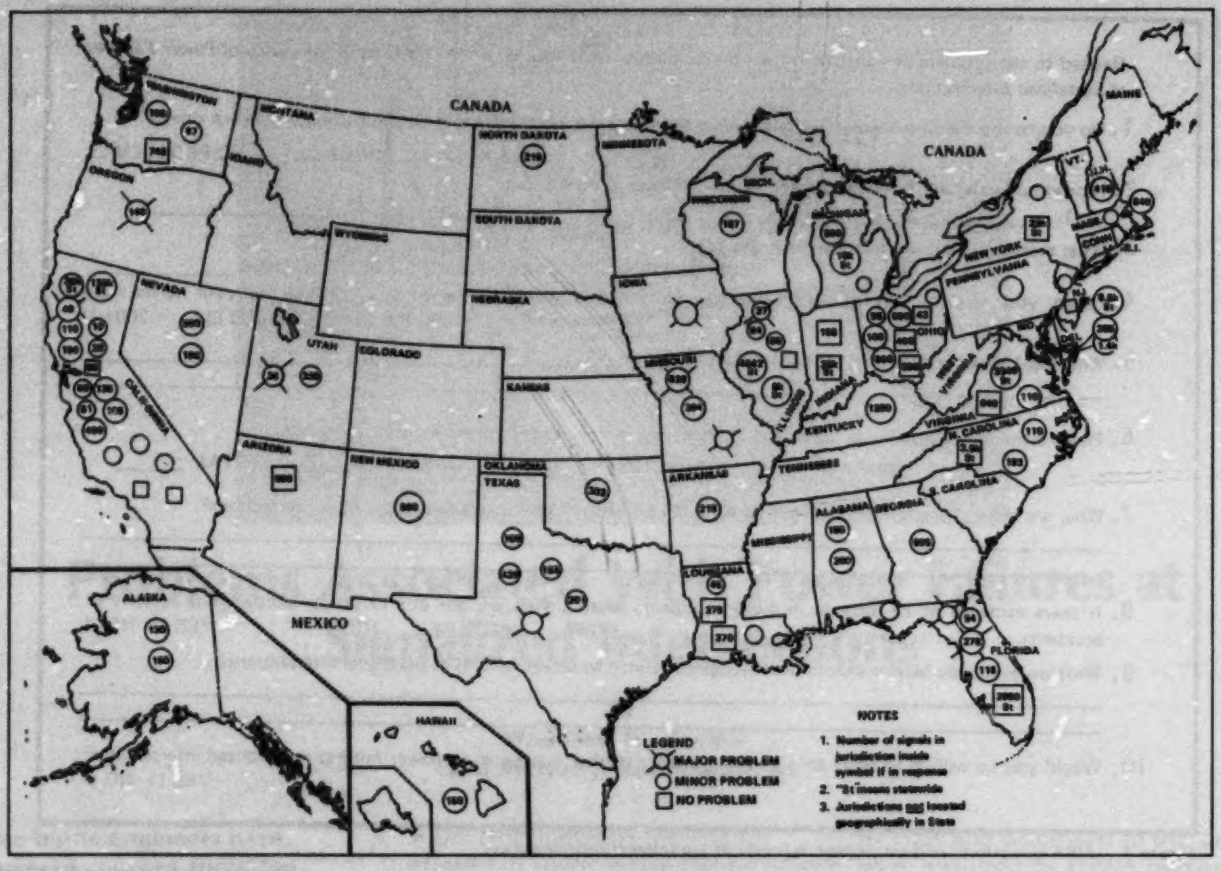

Figure 2.—Response distribution from HKA survey.

well as by human switching errors and equipment failures that result in outages cascading throughout a system.

HUFSAM concluded that areawide blackouts do not cause traffic safety hazards. Because the public readily perceives a problem, most drivers exercise caution.

Localized power disruptions

Localized power irregularities include the effects of lightning and high winds, unexplained surges (or "spikes") in normal power transmission, and small interruptions caused by switching lines or sources in power grids or networks.

Lightning may damage traffic signal equipment by a direct hit, by inducing voltage and current surges into wires and cables that feed into signal controls, or by causing the power supply to a signalized intersection to fail.

Some areas of the United States have more electrical storm activity than others. A National Aeronautics and Space Administration report contains an isoceraunic map of the United States, that indicates that the most active area is in central Florida, where thunder can be heard more than 100 days per year. The least active area is along the west coast, with about 5 days of thunder per year. A "flash density," which includes flashes between clouds and flashes to the ground, between 0.3 and 1.0 flashes per square kilometre (0.8 and 2.6 flashes per square mile) on each thunderstorm day affects traffic control equipment because changes in the density may induce surges in power and signal lines. (5)

Lightning often causes failure of the power supply to a controller and damage to detector amplifiers. To reduce traffic control equipment susceptibility to damage from induced surges, various lightning protection devices may be adapted to existing systems or installed on new systems. These devices are increasingly important with new, sophisticated systems involving

solid-state electronics. The new systems are more efficient than earlier systems, but more vulnerable to power surges.

Localized outages also occur when power transmission lines feeding nearby signalized intersections are damaged by wind or downed poles from vehicular accidents. The duration of such outages depends on the time it takes for a maintenance crew to correct the problem or for power to be rerouted in established grids. In some areas up to 50 percent of the outages are caused by irregularities in the normal power supply, resulting in surges that blow fuses or cause other equipment failure.

Frequency and duration

Frequency and duration of power outages, and the resulting signal outages, vary with the geographic area, the cause of an outage, and the type of signal equipment. Computerized systems can detect and report many outages that normally would not be noticed or recorded. Average frequency and duration figures therefore have little significance.

HUFSAM reported the following experiences:

- A Southwestern city traffic agency with 385 signalized locations reported an average of two localized power failures per month, lasting between 15 minutes and 2 hours. Most of these were caused by wind, electrical storms, or accidents.

- A Midwestern State agency estimated an average of one signal outage per signal per year from various causes, although the agency indicated that small outages were not a serious problem. The agency also indicated there may be power outages that are not reported. Many of these outages result from problems in power transmission lines that often are automatically corrected. In areas with a well designed power network these outages may be detected and corrected in 2 to 5 minutes by switching the power supply.

- A Midatlantic State agency reported an annual signal outage rate of approximately 20 percent of the number of traffic signals. These outages lasted 15 minutes to more than 3 days, were primarily caused by electrical storms, and usually affected only one signal per incident.

- A recent week's data from a Midatlantic city agency (where controllers are on a central computer system) indicated an annual outage rate of about 36 percent of the number of controllers. Duration averaged slightly over 35 minutes.

- A Florida county traffic agency with 1,760 signalized locations reported that in fiscal year 1978 traffic maintenance crews responded to 193 signal outages caused by power failures not reported by the power company. This accounts for about 1 percent of all trouble calls responded to. To measure the reliability of the area's power service in conjunction with the phasing in of central computer control for the traffic signals (about 45 percent complete), the agency investigated power supply outages (hits) reported by area computer facilities. One facility, discerning hits as short as one power cycle, reported an average of 18 hits per week with one-half lasting 0.5 to 30 seconds. Another computer facility that identified hits of 1 second and longer reported 73 hits over 1 year, 4 of which were major areawide power failures.

- A north-central city agency reported that in a recent 1-year period there were 231 power failures in a network of 720 signalized intersections: 103 were caused by local power company troubles (15 percent were weather related), 34 by vehicular accidents, and 94 by vandalism or contractor damage.

- A west coast city agency reported approximately six power failures per year lasting from 10 minutes to 4 hours and averaging 45 minutes to 1 hour. These failures were caused by electrical storms, equipment failures, and accident damage. Ten to twenty of the city's 650 signalized intersections may have been affected because of common power feeds.

- A large east coast city agency reported that individual signal outages occur annually on approximately 60 percent of all signalized intersections. For the same period the power company reported power failures on about 4 percent of the number of signalized intersections. Signal outages were caused by blown fuses, coordination failure, vandalism, mechanical failures, and defective traffic cables and power company cables.

Some rural areas experience an above average number of power failures that are not always caused by electrical storms. However, as power suppliers become more reliable, this is a decreasing phenomenon.

The number of contiguous signalized intersections affected by a power failure varies with the signal system and where the power fails in the system. Localized power outages usually affect 3 to 10 closely grouped signalized intersections, with 3 being the more common. More intersections may be affected when groups are tied to a common power feed (not usual practice) that fails or when there is an areawide outage.

Responses to HKA's questions support these HUFSAM findings. Only 10 percent of HKA's respondents considered power

failure to be a major problem, 72 percent a minor problem, and 18 percent considered power failure to be no problem. The estimated number of annual power failures at signalized intersections in the respondents' jurisdictions (question 4 in figure 1) is reported in table 1.

Table 1.—Annual power failures at signalized intersections

Estimated annual power failures	Respondents
	Percent
Less than 50	72
50 to 100	21
101 to 200	3
Greater than 200	4

The information collected by HUFSAM and HKA indicates that the annual number of signal outages from power failures that agencies can expect is from 10 to 100 percent of the number of signalized intersections. These outages last an average of 30 minutes to 1 hour.

Power Failures and Accidents

Statistical data for relating accidents to signal outages are either unavailable or unsuitable. This may be part of the reason few traffic engineers responsible for signal operation and maintenance regard signal outages as a severe accident problem.

A major obstacle in effectively linking signal outages and accidents is that most accident reports record (if at all) only that a traffic control device failed and that this contributed to the accident. For a signal outage accident analysis to be useful, the accident report narratives must be reviewed to establish whether a traffic control device failed because of a signal outage and if so, to what extent the outage contributed to an accident. Also, signal outage reports would have to be reviewed to determine if the outage was caused by a power failure.

Because the public is usually aware of areawide power outages, traffic engineers attending HUFSAM's workshop felt that areawide outages are not as hazardous as localized outages. Also, large outages usually accompany other conditions that cause the public to be more cautious.

The following signal outage accident experiences were reported by HUFSAM:

- A large east coast city agency determined that from January to June 1978 approximately 0.25 percent of accidents occurred where a traffic control device failed. Traffic control device failure could be anything from a downed stop sign to a malfunctioning traffic signal. Accident reports did not always indicate whether the traffic control device failed.
- Data from a west coast city agency indicated that over 2½ years approximately 25 percent of all accidents during signal outages occurred when the signal malfunction was caused by a power failure. Approximately 0.04 percent of all accidents were attributed to power failure signal outages.
- A Southeastern county agency's reports of accidents at signalized intersections during a power outage revealed that failure of a traffic control device directly contributed to only one-third of such accidents.
- A Midwestern State agency's computer codes all accidents to intersections whether they occur at the intersection or midblock. Accident reports would have to be analyzed manually to determine an accident-to-signal outage relationship.
- A Southwestern city agency reported no accidents during a 24-hour areawide power outage caused by tornadoes and flooding. This was attributed to public awareness and the public exercising caution during a crisis.

In HKA's survey, 70 percent of the respondents indicated that power failures did not increase traffic accidents. Two-thirds of the respondents who indicated there was an increase felt the accidents were not severe.

It may be extrapolated from the information above that less than 0.1 percent of accidents occur when a signal outage was a contributing factor. Less than 0.05 percent of accidents occur because of signal outages from power failure, and these accidents are not severe. Available data are insufficient to yield nationally representative statistics. The extrapolations merely indicate an order of magnitude.

Signal Outage Operating Procedures

Traffic agencies learn about signal outages from their staffs, police, citizens, or central computers if the signal system is computerized. Generally, signal outages are self-evident in blackouts. Response procedures include mitigating the hazard, correcting the signal outage, and notifying the public. Agencies that maintain signals themselves respond to signal outages with their own maintenance crews. Agencies that contract for maintenance work have the contractors respond to signal outages.

Mitigating the hazard

Most maintenance and repair vehicles carry portable temporary stop signs that are usually erected if this control is adequate for traffic flow.

At signalized intersections where traffic flow is critical, police usually direct traffic while the signal system is being repaired. One jurisdiction stated that it has a civilian force to direct traffic during emergency situations. Police coverage in some rural areas may be difficult to obtain because of limited staff. It also was noted that police sometimes must stop directing traffic to respond to a more important call.

A Florida county uses "failsafe" devices (fig. 3) on many of its signalized intersections and intends to phase in full application. These battery-operated flashing units are located in the signal head but operate independently of the standard signal bulb and power source. They are actuated when power to the signal controller is cut off. Some of the signalized intersections have an emergency generator as a backup for power outages.

A Southwestern city agency installs four-way stop signs if three or fewer signalized intersections are affected by power failure. Stop signs are not used if more than three intersections are affected because the situation is similar to an areawide outage where people readily perceive the problem. In addition, an outage involving more than three intersections is usually caused by a transformer problem that often can be corrected faster than the traffic agency can respond with alternate traffic controls at all affected locations.

An Illinois State law stipulates that intersections with signal outages automatically be considered four-way stops. At urban arterial intersections that do not function well as four-way stops because of traffic flow conditions, police must direct traffic. As a result of the legislation, using temporary stop signs at intersections with signal outages is rare.

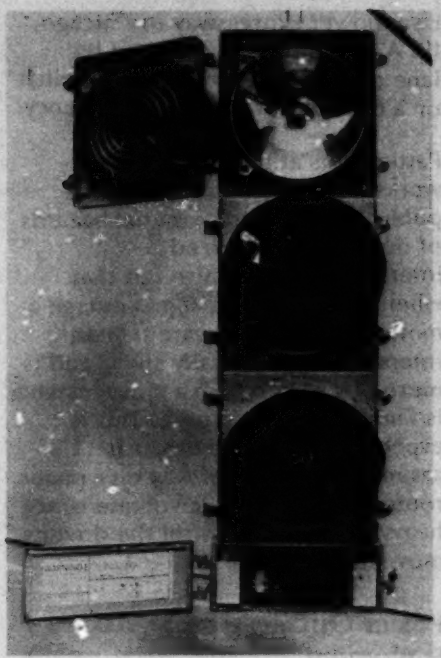

Figure 3.—"Failsafe" battery-operated traffic signal flashing unit. —courtesy of Metro Dade Department of Traffic and Transportation

Correcting the signal outage

A 1966 survey estimated that about 90 percent of State and city traffic agencies maintain and repair their own signals, while 10 percent contract for signal maintenance.[2] Standard operating procedures usually cover staff signal maintenance. Most contracted maintenance specifies a maximum allowable time to respond to an outage. In urban areas the time is typically 2 hours or less; longer times usually are specified in suburban and rural areas. Some agencies have service agreements with several contractors, providing for more flexibility. Agencies usually have a goal of less than 1 hour for response to an outage. Some urban areas indicate a normal response time of 30 minutes. An Indiana court action determined that 2½ hours was too long for response to outages so a closer repair unit was ordered. The court did not determine what the maximum allowable response time should be.

Agencies maintaining and repairing their own signals usually have crews on call for emergencies during offhours and weekends. In large cities crews may be on duty at all times. The goal of all traffic agencies is to be able to respond 24 hours a day to signal outages.

Planned outages at signalized intersections may require police control, stop sign erection, signal head bagging, or signal head removal, depending upon the expected duration of the outage.

Respondents to the HKA survey listed a range of options to use during power failures (question 7 in figure 1) including police control and several alternate power sources such as batteries, multiple power feed, solar, and generators. However, over 70 percent indicated that *no* measures were taken to solve the problem (question 2 in figure 1).

Notifying the public

Actions to notify the public often depend on how long the signal outage is expected to last, how critical the signalized intersection is, how widespread the power outage is, and the standard operating procedure of the traffic agency. Because areawide power outages are self-evident, no special notification efforts are needed.

[2] "Intersection Controller Survey," Battelle Memorial Institute, 1966. Unpublished.

In large urban areas, local radio stations often have helicopter surveillance of traffic during peak hours and broadcast traffic reports including information on signal outages. This information on outages is obtained by observation, direct communication with police, or monitoring police or citizen band (CB) radio.

Usually the public is notified of planned outages at critical intersections by radio or newspaper. In some cases residents have been notified by mail.

Traffic agencies with a central computer control system usually have a backup emergency power supply for computer operation. New York City's emergency power supply also powers radio and teletype equipment, providing direct teletype to radio stations. The traffic agency maintains special telephone numbers the public can call for emergency information.

It was noted in HUFSAM's workshop that CB radio, with its rapidly growing popularity, is providing more traffic information to the public. The Illinois Department of Transportation plans to study the potential of CB radios to determine the cause of a lane blockage on the Eisenhower Expressway in Chicago, Ill. A lane blockage detected by lane occupancy detectors will switch on a CB monitor at the traffic agency.

Participants at the HUFSAM workshop felt that informing the public reduces the potential hazards of outages at signalized intersections. They also felt that when approaching a signal outage a motorist who is unaware that an intersection is normally signalized is more of a hazard than a motorist who is familiar with the route. This is especially true if the driver is assuming the right-of-way on a major route and if the traffic volume is not heavy enough to control the driver's movement.

Power Outages and Signal Equipment

Although power restoration at signal outages may eliminate the potential for increased accident hazards and congestion, additional work is needed to restore some types of signal control equipment to normal operation. Restoration of any coordinated timing relationship of noninterconnected signal systems or the resetting of time clocks for switching signal cycles or offsets for multidialed isolated controls and interconnected coordinated systems often is required to regain traffic flow efficiencies at affected locations.

Followup maintenance to adjust timing is normally not required for isolated, single dial, pretimed controllers and most actuated controllers. It may be necessary, however, to reset the time clock mechanisms for isolated multidial, pretimed signal controllers and pretimed and background cycle actuated system-master controllers unless there is a backup spring drive for the time control. The adjustment is minor and can be made in a short time.

After a power failure outage, the adjustment of one or more of the controllers of a group of pretimed controllers that operate as a coordinated noninterconnected system is relatively simple but may require more than a nominal amount of time. Each affected controller must be checked and manually corrected to reestablish the proper time relationship between the zero or baseline time of the coordinated system and the start of the main street green.

The need for controller retiming after power failure signal outages is expected to decrease in the future. Intersection control and system control equipment that require these adjustments after a power failure are outmoded. As signal equipment is modernized and more sophisticated controls are installed, power failure signal outages are expected to have little, if any, adverse impact on traffic flow efficiencies after the power is restored and the signal resumes operation.

Conclusions

The consensus of the HUFSAM study and the HKA survey is that power failure at signalized intersections is considered by most traffic engineers to be a minor problem. The limited data indicate that annual power failure signal outages vary from 1 per signalized intersection to 1 per 10 signalized intersections, with the outage lasting an average of 30 minutes to 1 hour. The number and severity of accidents do not appear to increase significantly. Less than 1 accident in 1,000 occurs because of a signal outage and less than 1 accident in 2,000 occurs when power failure caused the signal outage.

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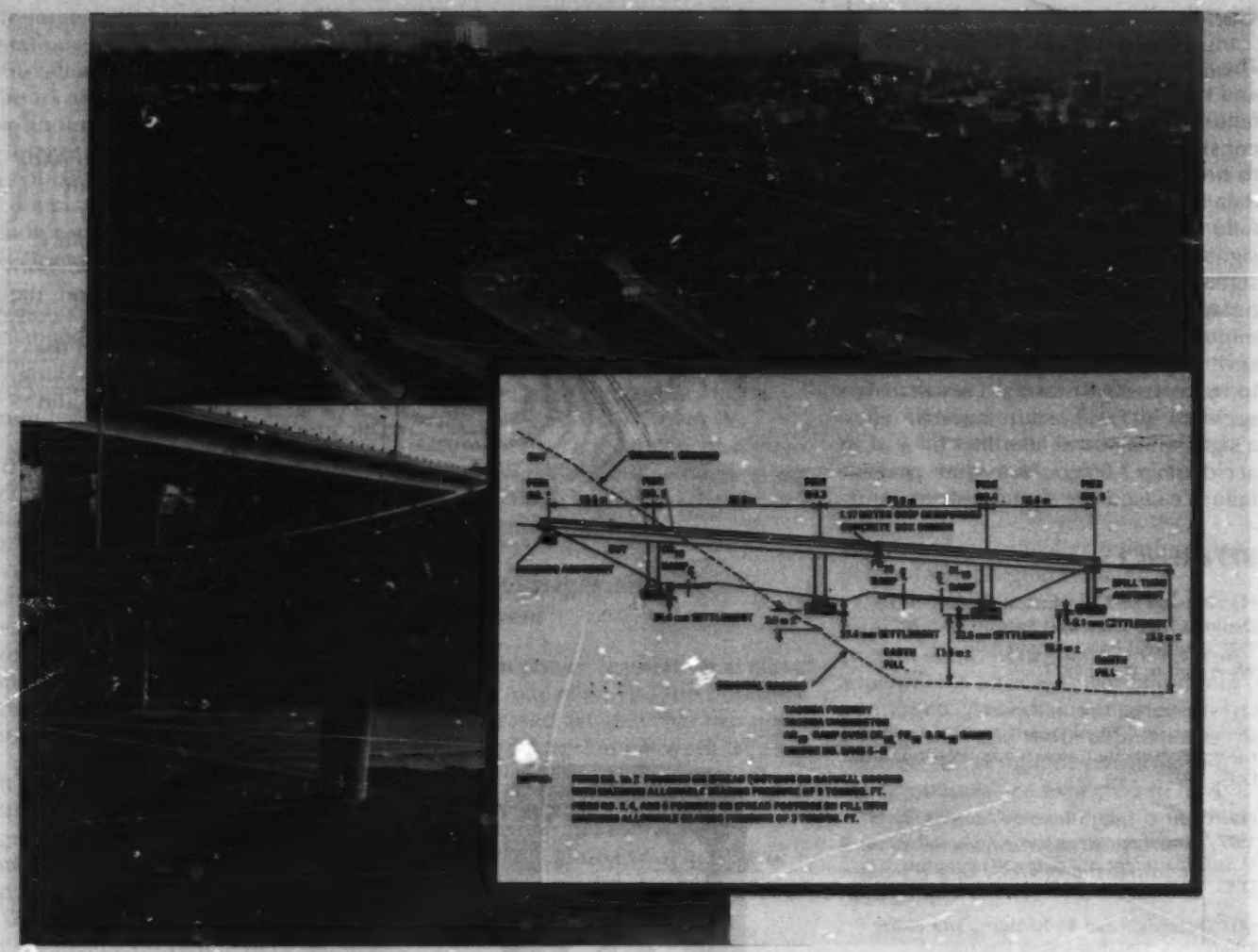
Woodrow W. Rankin is the Director of the Transportation and Safety Division of the Highway Users Federation for Safety and Mobility (HUFSAM). In his work with HUFSAM and its predecessor organization, the Automotive Safety Foundation (ASF), he has studied and researched the use of freeway air rights for parking, the management of the safety and traffic engineering functions in highway agencies, the allocation and apportionment of highway funds, the signing and marking of no-passing zones, and the selection of effective traffic signs based on field tests. Before joining ASF, Mr. Rankin was a traffic engineer in Wichita, Kans., Ottawa, Ontario, and Pittsburgh, Pa.

Merton J. Rosenbaum, Jr., is a highway research engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. He is involved in research to improve traffic safety and capacity on rural and urban highways. Before joining FHWA in 1961, Mr. Rosenbaum worked for 15 years with the Virginia State Highway Department in highway location and design, traffic and planning, and computer applications.



Tolerable Bridge Movements Introduction

by

Albert F. DiMillio

This article introduces a series of engineering research articles on current efforts to develop improved tolerable movement criteria for highway bridges.

This series will be published in the next three issues of *Public Roads* and will consist of the following parts:

- Part 1—Bridge Movements and Their Effects.
- Part 2—Analytical Studies of the Effects of Movements on Steel and Concrete Bridges.
- Part 3—Tolerable Movement Criteria for Highway Bridges.

Foundation movements under highway bridges have been predicted and measured; however, there have been few investigations of the effect these movements have on the safety and serviceability of the bridge structure. As a result, tolerable foundation movement has not been established beyond the conceptual stage.

The majority of highway bridge foundations are designed to prevent settlement. Many highway engineers believe such movement will cause significant superstructure distress and poor vertical alinement. Consequently, most bridges are supported by piles because they provide greater protection against settlement than do spread footings.

In many cases, a spread footing can adequately provide bearing capacity support and reasonable protection against excessive settlements at a much lower cost. When deciding whether to use piles or spread footings, the highway engineer must have an agreed upon definition of "excessive settlement" or more appropriately "tolerable movement" of various bridge types.

In addition to the disagreement on the definition of tolerable settlement, there is also disagreement on the accuracy of settlement predictions. As a result, many engineers still prescribe strict settlement constraints to guard against erroneous predictions. In many cases, the precautions are unrealistic and underestimate the accuracy of the prediction techniques, which are accurate within 10 to 20 percent on the average and are rarely more than 50 percent off the measured value. This series of articles will discuss only the tolerable movement aspects of highway bridge engineering.

Using piles does not guarantee that there will be little or no settlement, unless the piles are founded on hard rock. The nature of load transfer from the pile to the soil surrounding (side friction) and beneath the pile (tip bearing) requires the movement of the pile to mobilize the load bearing capacity of the soil material. The relative movement required to mobilize maximum frictional strength is approximately 6.35 mm (0.25 in), while the displacement required to mobilize the soil's shear strength under the pile tip is usually much larger than that value. If the allowable settlement is restricted to a smaller value than what is required to mobilize tip resistance, the load carrying capacity of the pile is artificially reduced to the level of side friction support. The support capacity available at the pile tip then becomes an additional safety factor.

Overly restrictive settlement constraints can reduce the supporting capacity of friction piles significantly. To comply with the prevalent requirements of zero or near zero settlement, the foundation engineer must reinforce his or her design by adding more piles, using longer piles, or increasing the diameter of the piles. The conservative effect of this design philosophy is evident by examining a load-settlement curve from a pile load test (fig. 1). A slight increase in the allowable settlement

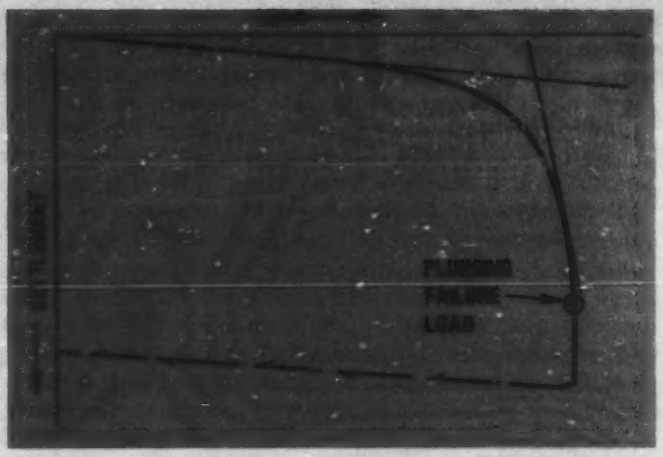


Figure 1.—Typical load-settlement curve from a pile-load test.

value can impact significantly the allowable design load for a pile foundation, especially within the straight line portion of the curve.

A relaxation of the stringent movement criteria also will impact significantly the use of spread footings whose major drawback is the risk of some settlement. Because the economic breakpoint in the normal design procedure for spread footings centers around 25.4 mm (1 in) of settlement, the majority of bridge engineers are reluctant to use this less expensive method of foundation support. As a result, the highway industry has been accused of having numerous "buried treasures of money" beneath bridge approach fills and intermediate piers because in some cases piles were used instead of spread footings or because more piles were used than were necessary. (*1*)[1]

Structural Consequences of Foundation Movement

Research indicates that bridge structures can withstand a reasonable amount of settlement and the amount of tolerable settlement varies according to the span arrangement (simple versus continuous) and length of span as well as other design variables. This indicates that the blanket criterion approach is inappropriate and the zero settlement design approach is too conservative. (*1-3*)

[1] Italic numbers in parentheses identify references on page 29.

An informal survey of tolerable movements was conducted to gather case histories that provide measured movement data and corresponding damage assessments. The information was enlightening but insufficient to establish rational tolerable movement guidelines. (*1, 2*)

Federal Highway Administration (FHWA) researchers studied the survey data and concluded that rational guidelines for tolerable movement could be developed through structural analysis if additional supporting field data could be obtained. A study was initiated in June 1978 with West Virginia University (WVU) to perform structural analysis studies and evaluate and expand the survey data. Functional distress and time rate effects also were assessed in the study.[2]

Foundation movements that induce dangerously high stress levels in structural elements are uncommon because the movements usually become intolerable for other reasons (such as poor rideability or esthetically displeasing) before the structural integrity of the bridge is threatened. If the movements occur slowly, the functional distress patterns usually will signal the need for corrective maintenance long before the danger point (structural collapse) is reached. To determine the various danger points WVU researchers analyzed various bridge types subjected to specific differential settlements at certain supports. Various span arrangements, span lengths, and girder depths were studied in both steel and concrete bridges.

During the same period, FHWA researchers studied the performance of approximately 150 highway bridges in Washington State that were supported by spread footings on compacted fills to determine if any damage had occurred as a result of settlement or other factors associated with the use of spread footings. In cooperation with the Washington State Highway Commission (WSHC), a circuit level survey was conducted on 28 of these bridges to obtain current elevations that could be compared with the as-built elevations to determine more precise movement data. The comprehensive case histories for the 28 bridges supported the survey data.

The FHWA researchers concluded that spread footings are a safe, reliable, and cost-effective alternative to pile foundations for the support of bridge piers and abutments on natural ground and engineered fills. Under certain conditions, such as potential scour at water crossings or when very difficult subsoil conditions are present, the cost differential between spread footings and piles decreases and the failure risk increases. Piles will always be in demand by the highway industry; however, resource economy requires the prudent use of spread footings where conditions permit.

All of the bridges inspected in the WSHC study performed well and in some instances caused fewer problems than nearby bridges on piles that had undergone differential settlement between the approach fill and the pile-supported abutment. The potential for a noticeable "bump" at the bridge end is much higher if the independently supported abutment does not settle with the approach fill, as it does with the spread footing that rests on top of the fill. Many of the bridges surveyed settled significantly without showing any signs of structural distress or deterioration of ride quality.

Although none of the bridges supported on spread footings on compacted fills experienced any distress or failures because of scour, one abutment was seriously damaged because of scour. The spread footing was supported on natural ground that gradually eroded because the soil was not adequately protected by riprap or other suitable measures.

[2] "Tolerable Movement Criteria for Highway Bridges," by West Virginia University. Not yet published.

Functional distress is more difficult to assess than structural distress because of its subjectivity. Functional distress is defined under this study as damage to the architectural elements or a reduction in ride quality. Architectural damage is less severe than structural damage that affects the integrity of a main supporting element of the bridge; however, architectural damage is usually more visible and causes an annoying or insecure feeling on the part of the motorist. It is also referred to as "cosmetic damage" and includes cracking or misalinement of bridge railings, curbs, decks, abutment wing walls, and damage to light poles and utility lines. The deterioration of ride quality involves the "bump" at the end of the bridge and other roadway unevenness associated with bridge foundation settlement.

The WVU researchers conducted structural analysis studies of the time-rate effects of foundation settlement to determine if certain bridge structures adjust more readily to a different shape if the settlement occurs slowly. For example, mitigating factors such as plastic flow in steel and long term creep of concrete were investigated.

Building codes in many major U.S. cities permit larger settlements than the American Association of State Highway and Transportation Officials (AASHTO) bridge specifications, even though building elements, such as glass doors and windows, elevator shafts, utility lines, and brittle wall panels, are more sensitive to foundation settlement than bridge elements. WVU researchers reviewed the research that resulted in improved building codes.

The proper definition of tolerable foundation movement will improve bridge design so that proper trade offs between structural design and foundation design can be made to insure that structural design is not optimized at the expense of the foundation system. The necessary guidelines for tolerable foundation movement were developed by WVU researchers in terms of settlement-induced stresses resulting from a predetermined amount of differential settlement acting on a particular superstructure configuration of a given stiffness. The design procedures package will be submitted to AASHTO for consideration and adoption.

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Albert F. DiMillio is a geotechnical research engineer in the Soils and Exploratory Techniques Group, Materials Division, Office of Research, Federal Highway Administration. Mr. DiMillio is project manager for FCP Project 4D, "Remedial Treatment of Soil Materials for Earth Structures and Foundations," and FCP Project 4H, "Improved Foundations for Highway Structures." Prior to his present position, he served as an area engineer in FHWA's Indiana Division Office and as the Regional Geotechnical Specialist in Region 5.

Optimizing the Expenditure of Maintenance Resources

by
Charles W. Niessner

Introduction

The cost of maintaining the Nation's highways is increasing by $300 million a year. The Federal Highway Administration (FHWA) has estimated that in 1979 State highway agencies spent $4.3 billion for highway maintenance.

In 1975 the Transportation Research Board under contract with FHWA recommended a program of high priority research and development maintenance needs. Twenty-eight needs were identified. The need having the highest priority was "Optimizing the Expenditure of Maintenance Resources." *(1)*[1]

The Implementation Division of the Office of Development, FHWA, initiated a series of studies to develop a methodology for meeting this need. Selected maintenance activities were analyzed using the techniques of value engineering (VE). Before 1975, VE was not widely applied in the highway field; only one State highway agency had a full-time VE staff. In 1975 FHWA awarded a contract to a private firm to conduct a national VE training program. During the training program, which is sponsored by the Office of Engineering and the National Highway Institute, FHWA, students work through the VE process by applying it to a real problem.

This article summarizes the Implementation Division's program to use VE techniques for maintenance activities. The program consists of 11 separate studies. Individual reports documenting the findings and recommendations of each study are or will be available.

Value Engineering Process

Value engineering is the systematic application of recognized techniques that identify the function of a product or service, establish a value for that function, and reliably provide the necessary function at the lowest overall cost. *(2)*

The value engineering job plan was established to perform the VE studies and to implement the recommendations of the studies. The key factors distinguishing the job plan from other methods used to solve routine engineering problems and reduce costs are analysis of function, use of specific, creative effort to develop alternatives, and maintenance of the performance needed by the user. Table 1 lists the nine phases of the VE job plan and the techniques used for each phase. *(2)*

[1] Italic numbers in parentheses identify references on page 34.

Table 1.--Value engineering job plan (2)

Phases	Techniques
Selection	Solicit project ideas
	Identify high-cost/low-value areas
	Plan the project
	Allocate resources
Investigation	Get facts from best sources
	Work with specifics
	Get all the available costs
	Challenge everything
	Identify the function
Analysis	Evaluate by comparison
	Put dollar value on specifications and requirements
	Put dollar value on key tolerances and finishes
	Put dollar value on key standards
Speculation	List everything
	Use creative techniques
	Deter judgment
	Do not criticize
Evaluation	Weigh alternatives
	Choose evaluation criteria
	Refine ideas
	Put dollar value on each main idea
	Compare
Development	Use search techniques
	Get information from best sources, specialists, and suppliers
	Consider specialty materials products and processes
	Use new information
	Consider standards
	Compile all costs
	Work with specifics
	Gather convincing facts
Presentation	Make recommendations
	Use selling techniques
	Be factual
	Be direct
	Give credit
	Provide implementation plan
Implementation	Translate plan into action
	Overcome problems
	Expedite action
	Monitor project
Audit	Verify accomplishments
	Report to management

In general, the job plan breaks down into the following elements: (1) Dividing an activity into its functions, (2) establishing costs for each function, (3) selecting the high cost functions and speculating on alternatives, (4) evaluating the most promising alternatives, and (5) developing and presenting the recommendations.

Efficient execution of the VE job plan requires a team effort—a maximum of five persons supported part-time by other staff in the organization. *(2)* Depending on the phase of the VE project, the study team size can vary from one person (presentation phase) to seven or eight persons (speculation phase).

Pilot Study

Snow and ice control was the topic selected for the pilot study to determine if the VE techniques could be successfully applied to maintenance activities. Because this topic proved too large for one study, the scope was reduced to materials handling and storage for snow and ice control. The pilot study was conducted by Colorado, Montana, Wyoming, and Utah in cooperation with FHWA.

Each State provided a two- or three-person team consisting of maintenance, research, and traffic engineers. The study lasted approximately 6 months, however, the actual time spent by the individual team members was considerably less. The study did not severely hamper the performance of the individuals' normal duties.

Three meetings were held in Denver, Colo., to organize and coordinate the study. The first session, an orientation meeting, included a discussion of VE techniques, limitations of the study, and the development of checklists for data collection. Because California was the only State highway agency with a VE staff when the pilot study was initiated, representatives of the California Department of Transportation were present at the first meeting to share their in-house VE experience. A second meeting was held about 1 month later to

review the checklists and coordinate the data collection. The final meeting, a "brainstorming" session, developed recommendations. At the conclusion of the study, a summary report was prepared by the FHWA program manager. *(3)*

The study's major recommendations concerned specifications, procurement practices, processing, and optimum storage. Many maintenance stations pay excessive amounts for unnecessary, high-grade materials because small purchases of low-grade materials are not economical. The study showed that large purchases from a centralized source can economically supply stations in a 40 to 48 km (25 to 30 mile) radius.

Ineffectively and inefficiently mixing salt and abrasives in the field was found to be a high-cost/poor production practice (figs. 1 and 2). The study indicated that mixing at the plant would reduce costs by 15 percent and greatly increase the uniformity of the mix.

It was also found that more expensive bagged (rather than bulk) salt was being used for no apparent reason. Using bulk salt could save money and probably reduce back injuries.

The estimated total annual savings in the four States from the implementation of only one of the recommendations (automatic mixing) is $220,000. The study costs of $60,000 compare favorably to the estimated minimum annual savings of $5 million if automatic mixing is implemented nationwide. VE techniques were proven effective in the analysis of maintenance activities, even in States with maintenance management systems in operation. *(3)*

Figure 1.—Spreading salt to be mixed.

Figure 2.—Sand and salt being mixed with a motor grader.

Continuing Studies

Because the pilot study results show substantial savings and the recommendations are applicable to other States, the program was continued. Ten additional maintenance activities were selected. Each maintenance activity is analyzed by three to five State highway agencies, with each State providing a two- or three-person team to conduct the study. A coordination meeting is held in each of the participating States to allow each State to see firsthand the other States' operations. A summary report based on the State reports is prepared and distributed by FHWA. Table 2 lists the studies that have been undertaken and the States involved.

Each summary report contains recommendations for revising the particular maintenance activity. These recommendations include changes in material purchasing procedures, use of new equipment or modifications to existing equipment, use of alternate materials, revisions in operating procedures, and training.

The following are a few of the more significant findings and recommendations from the various summary reports:

- In writing chemical compound specifications for traffic paints, the chemical components should be reviewed annually to determine the most cost-effective composition. The prices for chemical compounds are

Table 2.—Maintenance activities

Maintenance activity	States
Snow and ice control—materials handling and storage (*3*)	Colorado Montana Wyoming Utah
Snow and ice control—operations (*4*)	Colorado Utah Pennsylvania California
Shoulder maintenance (*5*)	Idaho Arizona West Virginia Iowa
Repair of continuously reinforced concrete pavement (*6*)	Texas Arkansas Louisiana Mississippi
Bituminous patching (*7*)	Oregon Utah Arkansas Pennsylvania
Sign maintenance (*8*)	Florida Arkansas Kentucky
Bridge painting (*9*)	New Jersey New York Wisconsin
Traffic striping (*10*)	Florida North Carolina Illinois New Mexico
Pavement joint repair (*11*)	Iowa Kansas Nebraska South Dakota
Rest area maintenance (*12*)	Alabama Michigan Illinois Virginia Wisconsin
Mowing operations (report to be published)	Alabama Florida Georgia

constantly fluctuating and in some cases it is possible to make a substitution without sacrificing performance or color. During the course of the study on traffic striping, one State revised its specifications and saved $211,000 when buying 1 155 m³ (305,000 gal) of paint.

- In rest area maintenance, labor accounts for 60 to 80 percent of the maintenance cost. Therefore, any reduction in employee-hours will reduce the overall cost. A review of current staffing plans resulted in the development of several alternatives that would accomplish the same level of service. Statewide annual savings from implementing one of these alternatives ranged from $247,000 to $395,000.

- The study on bridge painting found that the establishment of a flexible painting program that includes a general repainting cycle and an inspection program to identify precise repainting schedules for each structure could result in substantial savings. The establishment of this type of program can reduce or eliminate the need for extensive preparation of the steel prior to painting. If complete sandblasting of the steel is required, the cleaning costs can be as high as 60 percent of the total cleaning-painting costs.

- The use of ground control spreaders can save time, material, and fuel at a relatively low cost for the initial investment. Many existing hydraulic spreaders can be adapted to ground control systems for less than $1,500. Controlled tests have shown that these units can save up to 40 percent of the material that would normally be spread by nonground-oriented spreaders.

Although some of the findings apply only to those States that participated in the study, many of the changes can be applied in several States.

Summary

The total estimated cost for the VE studies was $500,000. The total estimated annual savings for only those States that participated in the various studies exceeds $8 million. Reducing maintenance costs will improve service because more of the activity or other activities can be undertaken.

Thirty-six training workshops have been conducted for both FHWA and State personnel. These workshops and the maintenance studies have induced several other States to initiate their own VE studies. Several State highway agencies have also established a full-time VE staff.

This program has demonstrated that VE techniques can be successfully applied to maintenance activities. Also, the program gave State maintenance engineers the opportunity to meet and discuss mutual problems and share information with personnel from other States. These meetings resulted in potential solutions for several problems not directly related to the study such as snow removal equipment and policies, mowing policies, and signing and traffic control. Other special purpose maintenance items were observed such as a small trailer for carrying warning signs and traffic cones to a worksite and an air deflector mounted horizontally above the tailgate of a sand spreader truck to prevent snow from accumulating on the taillights.

It is essential that every effort be made to optimize the expenditure of maintenance dollars. It is imperative that, where applicable, each State highway agency implement recommendations from other studies and analyze in depth its high-cost maintenance activities.

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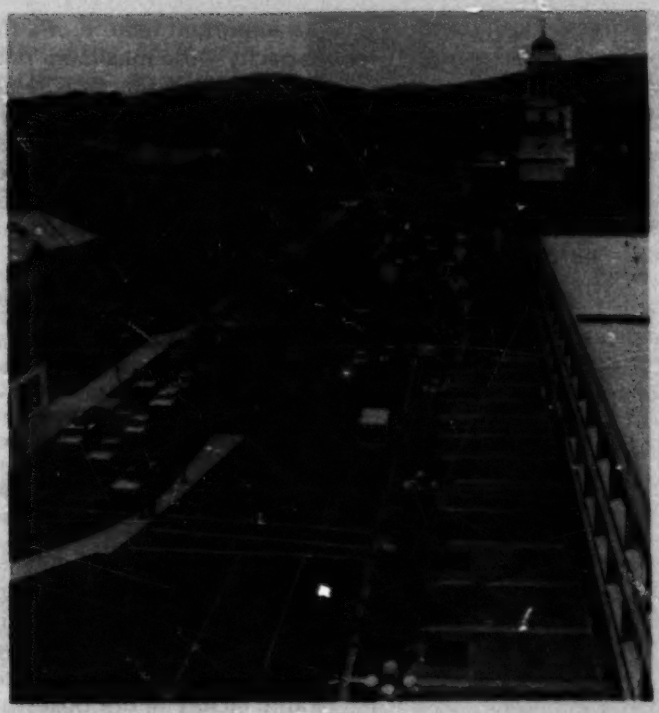
Charles W. Niessner is a highway engineer in the Implementation Division, Office of Development, Federal Highway Administration. His current responsibilities include the Implementation Division's delineation and value engineering programs.



Capacity and Quality of Flow on Urban Arterials

by
Guido Radelat and Gary Euler

This article describes a proposed process for determining the capacity and quality of flow on urban arterials. Many of the ideas presented here originated in the first phase of a study on the quality of flow in urban arterials, sponsored by the Federal Highway Administration (FHWA).

This article discusses arterial capacity and the influence of capacity controlling features that are used to define a section of arterial capacity. This sectional concept is proposed as an extension of the cross sectional or point measure currently used in the Highway Capacity Manual. *(1)*[1] A procedure for computing arterial capacity is also described.

The designation of levels of service for an urban arterial as a function of newly selected measures of effectiveness (MOE) is proposed. The correlations among these MOE's are discussed as well as their implications on the definition of levels of service. Estimating MOE's from independent arterial characteristics is described and the effect of one of these characteristics, signal progression, on the arterial MOE's is analyzed.

The article concludes with a discussion on updating the Highway Capacity Manual.

Introduction

An arterial street's ability to function as a traffic facility can be described by conventional measures of effectiveness (MOE). MOE's such as traffic volume, spot speed, and intersection delay apply to a *point* on the arterial; others, such as travel in vehicle-kilometres (vehicle-miles), travel time, and total delay, are related to a *section* of the arterial. The section MOE's determine the performance of a section of arterial whereas the point MOE's localize possible deficiencies.

[1] Italic numbers in parentheses identify references on page 43.

On arterials and other traffic facilities, both the quality and the quantity of traffic flow must be determined. The quality of traffic flow is determined by MOE's such as spot speed, intersection delay, travel time, total delay, travel time variance, number of stops, number of accidents, vehicle emissions, and fuel consumption per vehicle. The quantity of flow, on the other hand, is determined by MOE's such as traffic volume and vehicle-kilometres (vehicle-miles) of travel.

The following example illustrates the importance of determining the quantity as well as the quality of arterial flow. Suppose that a new traffic management strategy improves the quality of flow of an arterial section but does not change the quantity of flow. Obviously, the traffic performance of the arterial has been improved. If, however, the improved traffic conditions attract new traffic to the arterial, the quality of flow may be degraded to its previous level while the quantity of flow increases significantly. However, traffic performance on the arterial still has been improved, because more traffic is now being served with the same quality of flow as before.

Some of the quality of flow MOE's are related to the speed of the traffic flow: Spot speed, intersection delay, travel time, and total delay. Other quality of flow MOE's, such as travel time variance and the number of stops, indicate the flow's smoothness or jerkiness. The number of accidents MOE demonstrates the safety of traffic flow. Other MOE's—vehicle emissions and fuel consumption—relate traffic flow to social goals such as reducing air pollution and conserving resources.

One problem inherent in using arterial MOE's is accounting for trade offs among the MOE's. A potential solution is to combine the MOE's into a composite MOE. This is difficult, however, because individual MOE's are expressed in different units that cannot be converted easily into a single unit. Also, individuals may perceive the importance of a given MOE differently, which makes it difficult to rate individual MOE's when calculating the composite. For example, a driver is not directly interested in quantity of flow MOE's, but is primarily concerned with the speed MOE's and the jerkiness of the flow, which increase or decrease the driving effort. On the other hand, quantity of flow MOE's are important to traffic engineers and other public officials. For people living or working near a traffic facility, vehicle emissions and noise may be the most important MOE's. Finally, fuel consumption has become an important MOE for everyone because of the energy crisis.

An important feature of a traffic facility is its capacity—the maximum number of vehicles the facility can handle. Capacity is related to the quantity of flow MOE's and is often the only measure used to indicate a traffic facility's effectiveness. Another important feature of a traffic facility is the level of service (LOS)—the quality of service a facility can provide under specific conditions for a certain quantity of flow.

Capacity

The 1965 Highway Capacity Manual (HCM) defines capacity as follows:

> Capacity is the maximum number of vehicles which has a reasonable expectation of passing a given section of a lane or roadway in one direction (or in both directions for a two-lane or three-lane highway) during a given period of time under prevailing roadway and traffic conditions. (1)

In this definition, capacity is a unique volume—the maximum number of vehicles under prevailing roadway and traffic conditions at a given section of a lane or a roadway; thus it is a cross sectional measure, not a longitudinal one. Capacity corresponds to a point on the traffic facility under observation, and therefore, is defined in the HCM as a point MOE (fig. 1).

Figure 1.—1965 HCM concept of capacity focuses on the capacity of a point or single intersection.

When analyzing an arterial, however, the primary question is the flow along the arterial (fig. 2). The number of vehicles that can pass through an arterial section is more important than the number that can pass by a given point. In the first phase of a recently completed study on the quality of flow on urban arterials, the point concept of capacity was extended to apply to a section of an arterial. (2) The proposed definition of capacity follows:

> Arterial capacity is the maximum number of vehicle-miles of travel which can reasonably be expected to be accommodated by a given arterial segment (or section) per unit time and length under prevailing roadway and traffic conditions. This capacity can be expressed in either one-way or two-way flows. In the absence of time and distance modifiers it would be expressed in vehicle-miles of travel per hour per mile. (2)

In this definition, capacity is expressed in vehicles per unit of time (travel units divided by the length of the section under observation). Capacity is expressed in the same units in the HCM definition; however, the units of the HCM definition refer to a point, whereas the units of the proposed definition apply to an arterial section and constitute an average volume over the section.

To compute arterial capacity, a segment is defined as the zone of influence of a bottleneck or capacity controlling feature, and a section is a series of continuous segments. A capacity controlling feature is an element of the arterial whose point capacity is considerably lower than the point capacities along the portion of arterial upstream of it. It is generally an intersection, but occasionally may be a midblock point with a capacity lower than that of an intersection downstream.

In the study on quality of flow on urban arterials, segment capacity is defined as the maximum segment volume multiplied by the length of a segment, that is, the point capacity of the downstream capacity controlling feature times the length of the segment. (2) Segment capacity is expressed in vehicle-kilometres (vehicle-miles).

The segment capacities of a longitudinal section of arterial with six segments are shown in figure 3. The area under the segment capacity profile line—determined by the point capacities of the capacity controlling features—represents the segment capacity.

The capacity of a section is also determined by the arterial volume profile—the profile of current traffic volumes at points along the arterial section (fig. 3). For each segment, the shaded area under this line is the travel in the segment under prevailing conditions. For the conditions shown, the arterial section is not operating at capacity.

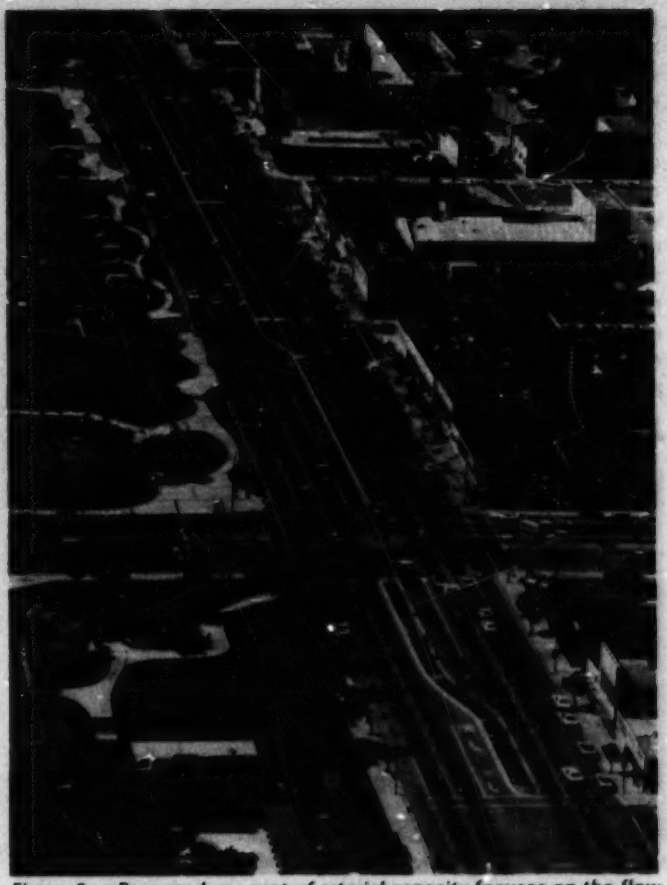
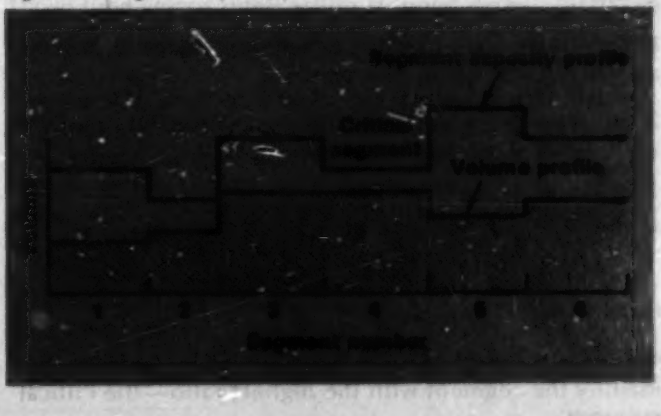


Figure 2.—Proposed concept of arterial capacity focuses on the flow along an entire length of an arterial.

Figure 3.—Segment capacity profile.



To compute arterial capacity, the travel/capacity ratio must be known for each segment in the section being observed. This ratio is found by dividing the segment

travel by the segment capacity (both quantities expressed in either vehicle-kilometres or vehicle-miles). The segment with the highest travel/capacity ratio is then identified as the "critical segment" (segment 4 in figure 3). The arterial capacity of the section is found by determining the total travel that will result from expanding travel in the critical segment to its capacity with the same proportional expansion of travel in the other segments; that is, travel is expanded, keeping the same travel ratios among the segments. Thus prevailing road and traffic conditions are maintained. This expansion is shown in figure 4. The volume profile resulting from this expansion—the arterial capacity profile—is also shown in figure 4.

Figure 4.—Arterial capacity profile.

The procedure for computing arterial capacity is as follows:

1. Determine the capacity controlling features along the arterial section being observed.

2. Measure traffic volumes and compute point capacities at these locations in vehicles per hour.

3. Compute travel and segment capacity for each segment, multiplying the volume and point capacity at the downstream capacity controlling feature by the segment length (results in vehicle-kilometres [vehicle-miles] per hour).

4. Compute travel/capacity ratios for each segment and identify the segment with the highest ratio—the critical segment.

5. Expand travel in the critical segment to its capacity and expand travel in other segments by dividing their travel by the travel/capacity ratio of the critical segment (results in vehicle-kilometres [vehicle-miles] per hour).

6. Find the arterial capacity of the section by adding the expanded segment travel figures and dividing the sum by the length of the section (results in vehicles per hour).

This definition of arterial capacity assumes that the turning movement percentages remain constant. This is not always true, but it is a reasonable assumption. There still may be unused segment capacity when an arterial section is operating at capacity. The above procedure will identify the wasted capacity and may suggest changes in vehicle routing to better utilize segment capacity.

Levels of Service

Capacity is essentially a quantity of flow MOE indicating the maximum quantity of flow that a traffic facility can handle. It can also express indirectly the quality of flow because for the same traffic demand, the higher the capacity of the facility, the better the service that can be expected.

However, it is important to know the quality of traffic flow, which can be determined by several MOE's, when a facility is operating either at or below capacity.

The 1965 HCM uses levels of service (LOS) from A through F to designate the quality of flow and defines LOS as follows:

> Level of service is a term which, broadly interpreted, denotes any one of the infinite numbers of differing combinations of operating conditions that may occur on a given lane or roadway when it is accommodating various traffic volumes. Level of service is a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. In practice, selected specific levels are defined in terms of particular limiting values of certain of these factors. (*1*)

LOS must be defined in terms of MOE's. The HCM uses three MOE's to determine the LOS of urban and suburban arterials: Average overall travel speed, load factor, and service volume/capacity (V/C) ratio.

The average overall travel speed is a good measure of arterial performance because it reflects the influence of local bottlenecks and the effectiveness of signal coordination. If measured for each segment of an arterial section, it can show where deficiencies exist.

The load factor determines the LOS of individual intersection approaches. Load factor is the ratio of the number of loaded green phases on an approach during a certain period of time to the total number of green phases available on that approach during that time. A phase is loaded when vehicles are ready to enter the intersection in all lanes during the entire phase without

"exceedingly long spaces between the vehicles at any time due to lack of traffic." (*1*)

The maximum possible value of the load factor is 1. However, an intersection approach can offer various levels of service while the load factor is at its maximum value. Also, the load factor loses its meaning as a measure of flow quality when an intersection is controlled by traffic-actuated signals or by no signal at all. Furthermore, the load factor cannot assess the quality of flow at midblock locations.

The V/C ratio is also used in the HCM to define the LOS at individual intersection approaches. It is actually a quantity of flow MOE that indicates how loaded a facility is. It can only be used to determine the LOS if a relationship is developed deductively between the V/C ratio and LOS or between the V/C ratio and a quality of flow MOE such as speed. However, it is not feasible to show any typical speed-volume curve for urban arterials. (*1*) An attempt to determine such relationships was unsuccessful because of the effects of other variables such as signal timing, turning percentages, and interference from parking maneuvers. Therefore, the V/C ratio does not seem to be an adequate MOE for LOS.

The limitations of the MOE's used in the HCM became evident during the study on quality of arterial flow. (*2*) This study included a literature review to identify all MOE's suitable for defining LOS. These MOE's were systematically analyzed and the following list of quality of flow MOE's was selected:

- *Travel time per kilometre (mile).* This speed MOE is equivalent to the average overall travel used in the HCM and reflects the overall performance of the entire arterial section under observation or each of its segments.
- *Intersection delay.* This point MOE measures the effect of individual intersections on the quality of flow and is a hybrid between a speed and a smoothness of flow MOE.
- *Intersection stops.* This point MOE indicates the influence intersections have on the smoothness of flow.
- *Travel time variance.* This segment MOE measures the overall smoothness of flow along an arterial section.

In addition to the quality of flow MOE's chosen for estimating the LOS, service volume, a quantity of flow MOE directly related to capacity, was selected. This MOE can be either a point or segment MOE and is expressed in vehicles per hour, vehicles per lane per hour, or vehicle-kilometres (vehicle-miles) per hour. As applied to a section, service volume is not really a measure of volume but of travel.

In the opinion of the authors, four MOE's are still too many for defining LOS. As discussed earlier, the difficulties of combining them are considerable. Also manipulating four LOS MOE's would be unduly complex and cumbersome for practical application. Furthermore, there is a significant correlation among the quality of flow MOE's; therefore, once the key MOE's have been identified, little meaningful information can be obtained from the others.

Several years ago, over 4,000 floating car runs were made on forty 1.6 km (1 mile) arterial sections in Washington, D.C.[2] Travel time, stopped delay at intersections, and number of stops over each arterial section were among the variables observed. The authors recently performed a regression analysis on the data collected for each MOE, and the following coefficients of linear correlation were obtained:

Number of stops versus intersection stopped delay	0.86
Travel time versus intersection stopped delay	0.84
Travel time versus number of stops	0.80

Similar conclusions were drawn in a study of the influence of various urban driving conditions on automobile fuel consumption. (*3*) A test vehicle was driven 383 km (238 miles) through nine routes in the Detroit, Mich., metropolitan area. Fuel consumption, speed, acceleration, and other travel-related variables were recorded. The statistical analysis of the data collected produced, along with many other results, the following coefficients of linear correlation:

Number of stops versus intersection stopped delay	0.79
Travel time versus intersection stopped delay	0.86
Travel time versus number of stops	0.85

In a study to develop a technique for measuring delay at intersections, field data were collected photographically at 10 intersections in four metropolitan areas. (*4*) An analysis of the correlation between the percent of vehicles stopping at an intersection and the stopped time per vehicle yielded a coefficient of linear correlation of 0.72. However, in a regression analysis of the percent of vehicles stopping and the logarithm of the approach delay per vehicle[3], a coefficient of linear correlation of 0.94 was found.

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[2] "A Method for Evaluating the Efficiency of Traffic Operations in a Signalized System," by G. Radelat, J. Raus, and F. A. Wagner. Paper presented to Committee TO-7 of the Transportation Research Board, January 1966.

[3] Time spent by the vehicle in traversing the intersection approach minus free flow time.

In summary, the LOS of an arterial might possibly be defined as a function of a section MOE that provides a measurement of speed, such as travel time, and a point MOE indicating the smoothness of flow, such as intersection stops or intersection stopped delay. Furthermore, the possibility that only one MOE may be used to define LOS is not discarded.

Estimating MOE's

Direct measurement of arterial MOE's is generally costly and time consuming. Also, in planning and design work MOE's are not measurable because the desired conditions do not exist in the field. Nevertheless, relationships exist between the MOE's and road, traffic, and environmental characteristics that may be measured or predicted. Knowledge of these relationships would make it possible to estimate the MOE's and then compute point capacity and quality of flow as a function of the MOE's. Also, the effect of variations in selected characteristics on the MOE's could be estimated. The HCM is used to perform this type of estimation based on relationships between two or more variables developed from empirical methods.

Independent characteristics

In the study on quality of arterial flow, independent road, traffic, and environmental characteristics related to either capacity or quality of flow were analyzed. (2) The study research team then identified which of these characteristics influence the point MOE's associated with an intersection and which influence arterial segment MOE's (table 1). A subset of these characteristics will be used in the proposed HCM to estimate the selected MOE's, which in turn will be used to predict point capacity and quality of flow along an arterial.

Procedures to relate characteristics to MOE's

The conventional procedure to develop relationships between independent road, traffic, and environmental characteristics and the MOE's for a traffic facility is to observe simultaneously these variables in the field and then perform a statistical analysis on the observations. It is possible, however, to augment the field data from this empirical procedure with information generated from "logical" relationships between the variables. For example, the position, speed, and acceleration of a vehicle following another one is related to the position, speed, and acceleration of the vehicle being followed; also, a left-turning vehicle is not delayed if no opposing vehicle approaches during a time equal to or greater than the acceptable gap. These logical relationships are inherent in many analytical and simulation models; therefore, these models can supplement empirical information.

Table 1.— Independent road, traffic, and environmental characteristics related to capacity or quality of flow (2)

Related to Intersection Point MOE's	
1. Approach width	11. Progression
2. Number of lanes	12. Total green per approach
3. Lane control	13. Number of times green
4. Median type	14. Volume per movement
5. Type of bus stop	15. Truck volume
6. Right turn on red	16. Local buses
7. Parking condition	17. Bus dwell time
8. One/two way	18. Lane blockage
9. Parking turnover	19. Pedestrian levels
10. Distance to first parked vehicle	

Related to Arterial Segment MOE's	
1. Width	8. Parking turnover
2. Number of lanes	9. Segment demand
3. Median type	10. Truck volume
4. Length	11. Local buses
5. Bus stop type	12. Bus dwell time
6. Parking condition	13. Lane blockage
7. One/two way	

Traffic simulation models, such as NETSIM (5), are useful in analyzing intricate urban arterial situations because they combine the empirical and logical elementary relationships needed to define more complex relationships. Nonetheless, several statistical problems arise when combining field data with simulation data. These problems must be resolved to fully use simulation to establish the necessary relationships.

Progression

One of the independent characteristics that is difficult to measure is traffic progression along an arterial. Progression is related to traffic signal coordination and has been measured by the width of the progression band that goes through the green intervals in a time-space diagram. This measure is not very accurate because it depends on the chosen progression speed and the lengths of the straight portions of the band. Also, it does not take into account vehicle arrival distribution and the secondary flow that turns onto the arterial.

A simpler measure is the number or proportion of vehicles that arrives during the green interval. This measure has a unique value, it can be measured at each intersection, and it accounts for secondary flow from side streets. The measure loses accuracy when vehicles join a queue during the green interval because even though these vehicles are counted as arriving on green, they must actually stop before proceeding through the intersection. This would be reflected in MOE's such as stops, delay, or travel time. Previous research, however, concluded that ". . . delay is a linear function of the decimal fraction of vehicles arriving on red." (6) This conclusion, which was reached analytically, was tested by the authors using the NETSIM computer simulation model.

A hypothetical arterial having three intersections was constructed to study the effects of signal progression on two links. All approaches on the main arterial were one lane, with left turn pockets at each intersection. The selected free flow speed was 55 km/h (34 mph) and the intersections were 450 m (1,477 ft) apart. All approaches on the cross streets were also one lane. Speeds on the cross streets varied from 40 to 55 km/h (25 to 34 mph) for the three alternatives studied.

The alternatives were constructed by varying volumes on all approaches (both on the main arterial and on the cross streets) and by varying the turning percentages from the cross streets onto the main arterial. Cycle length and proportion of green times were different for each alternative, but were constant within each alternative being studied. The signal offset between adjacent intersections was then varied in 10-second increments. The timings used were not necessarily optimum, although judgment was used in their formulation insofar as splits were proportional to approach demands and larger cycle lengths were used for higher demand alternatives.

The proportion of vehicles arriving on green (PROGREEN) was calculated using NETSIM. This was then compared with the MOE's—average speed on the link, average delay per vehicle, and average number of stops per vehicle—derived for the corresponding alternatives by NETSIM. The resulting regression lines and multiple correlation coefficients are shown in figures 5, 6, and 7.

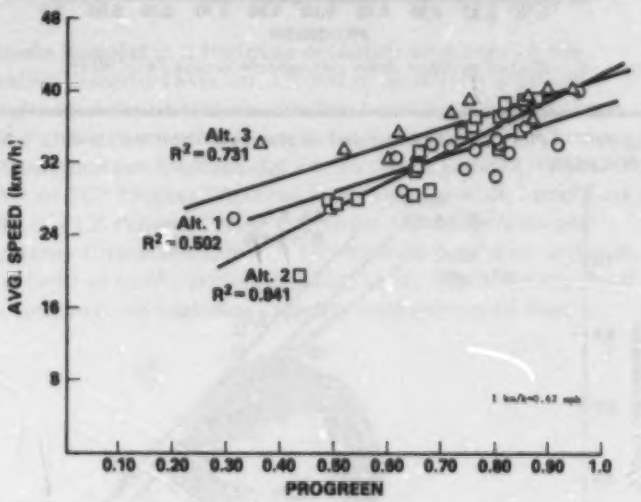


Figure 5.—Plot of link average speed versus PROGREEN.

These figures show that although a linear trend exists in all of the comparisons, a single relationship does not exist and the PROGREEN variable is not consistently correlated with any of the other MOE's. This is because of the following influences that confound the relationship between the PROGREEN variable and the traffic MOE's:

- Demand—Higher demand can produce higher delays for the same value of PROGREEN. For example, longer cycle lengths are normally used in higher demand situations. For the same values of PROGREEN, the average time that vehicles will be stopped will be longer (assuming the same split) than in a lower demand situation when a shorter cycle length is used.

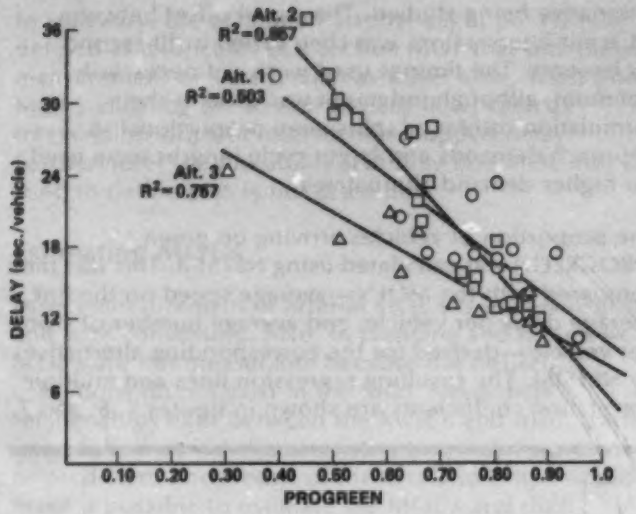
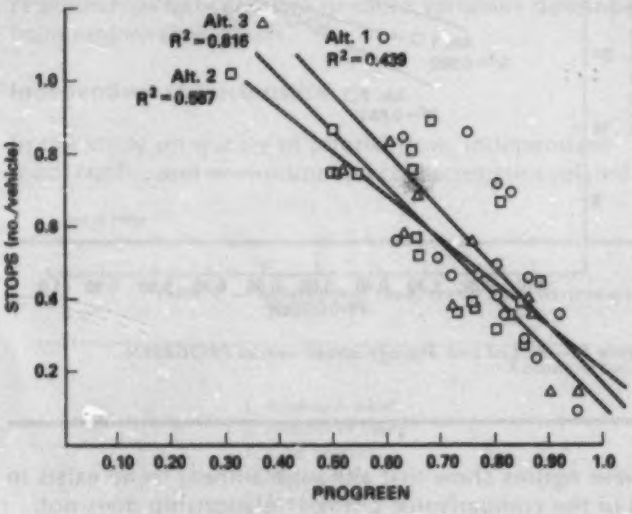


Figure 6.—Plot of average delay per vehicle versus PROGREEN.

Figure 7.—Plot of average number of stops per vehicle versus PROGREEN.



• Residual queues—A platoon of vehicles that overlaps the end of the previous red interval may produce the same value of PROGREEN as a platoon that overlaps the beginning of the next red interval. However, the average number of stops per vehicle will be higher in the former case because the vehicles that arrive on red as well as the vehicles that arrive at the beginning of the green will stop until any residual queue has dissipated. The residual queue could also be caused by cross street traffic from an upstream intersection. This may not make much difference in average delay or travel time, but it can make a big difference in the average number of stops per vehicle.

The conclusion is that the PROGREEN variable indicates the quality of signal progression, but other influences, particularly demand, must be taken into account before relationships between PROGREEN and the traffic MOE's can be established. Perhaps the best way to accomplish this is by computer simulation where an extensive, controlled study can quantify the effect of demand or other influences on the relationships between PROGREEN and the MOE's.

Finally, it should be noted that although PROGREEN indicates the quality of progression, it will not pinpoint the problem—a faulty cycle length, split, or offset—if the progression is poor.

Estimation of Capacity and Level of Service

According to the approach outlined in the study on the quality of arterial flow, an arterial section's capacity may be calculated by first measuring or estimating the section's road, traffic, and environmental characteristics that significantly affect arterial capacity. (2) Using these relationships, the maximum service volume or point capacity of the capacity controlling features can be computed. Segment capacity and arterial volume profiles can then be sketched and arterial capacity estimated.

A similar method can be used to calculate the quality of flow for a given service volume. If the road, traffic, and environmental characteristics and the relationships between them and the quality of flow MOE's are known, these MOE's can be computed for each segment. Then, based on the value of the MOE's, the LOS of each segment as well as that for the entire arterial section can be determined.

Many relationships between independent characteristics and MOE's have been formulated in the development of the HCM. However, more work is needed in this area to reflect changes in vehicle and driver behavior since 1964 when data for the HCM were collected, to define more precisely relationships that were developed using insufficient data, to reflect new developments in the traffic engineering field, and to respond to new concepts originated in the first phase of the study on quality of flow in urban arterials. (2)

Research to define the characteristic-MOE relationships will be done in the second phase of the FHWA study. A process will also be developed to express LOS in terms of the chosen MOE's. It has been recommended that a panel of experts on arterial capacity determine what that process should be.

The National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board is conducting a study on urban intersection capacity that will contribute to the understanding of arterial capacity and quality of flow. By 1982 or 1983, other research planned and conducted by the FHWA and NCHRP on different aspects of highway capacity will produce the material necessary to prepare the improved, proposed Highway Capacity Manual. The new manual is expected to be a valuable guide for planners, designers, and traffic engineers for the rest of this century.

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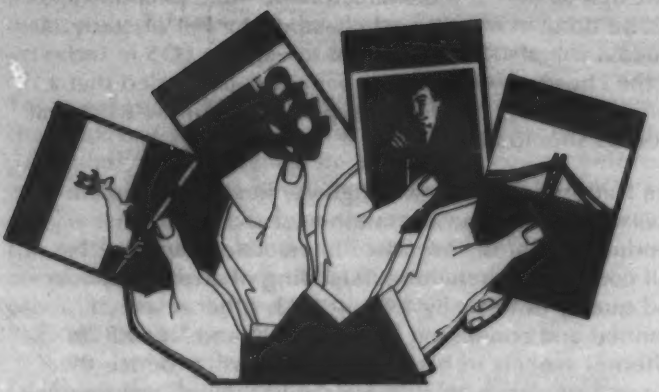
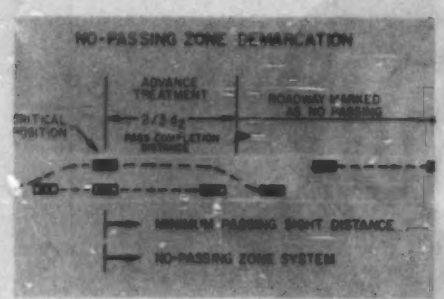
Guido Radelat is a highway research engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been with FHWA since 1964 and is currently manager for Task 2K2 of FCP Project "Metropolitan Multimodal Traffic Management," Task 2M1 of FCP Project "Arterial Flow and Control," and Task 9B2 of FCP Project "New Concepts Development and Systems Characterization." Dr. Radelat has been active in the field of traffic simulation and lately has been involved in research on highway capacity and quality of flow.

Gary Euler is a highway research engineer with the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been in his current position for 3 years and has worked principally in the areas of signal control, model development, and strategy evaluation. Mr. Euler previously was employed as a planner with the Metropolitan Dade County Planning Department in Miami, Fla.



Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Division. The reports are available from the address noted at the end of each description.



Passing and No-Passing Zones: Signs, Markings, and Warrants, Report No. FHWA-RD-79-5

by FHWA Traffic Systems Division

This report gives suggested criteria, warrants, and traffic control devices for designing and designating passing and no-passing zones on rural two-lane highways. The criteria are based on vehicle performance during a passing maneuver, the premise that sight distance should be provided at the critical position where the passing vehicle and passed vehicle are abreast, and evaluation of traffic control devices in providing the required visual information at this critical position. Minimum passing sight distance is the sight distance required at the critical position to permit a passing driver to perceive an opposing vehicle at a sufficient distance to safely complete the pass.

An advance pavement marking system is proposed consisting of a dotted yellow centerline throughout the pass completion distance and a no-passing zone pennant sign located at the beginning of the no-passing zone. This will advise a driver that a pass should not begin beyond the start of this marking system because sufficient distance will not be available to complete the pass before reaching the no-passing zone. An economic analysis indicates a benefit/cost ratio of 6.1 if the system were to be implemented nationwide.

The use of the single solid yellow centerline on rural two-lane highways to designate "restrictive" passing zones is evaluated based on driver understanding tests, accident rate comparison, and opinions of engineering, enforcement, and legal personnel on expected safety, enforcement, and compliance problems.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 80 114564).

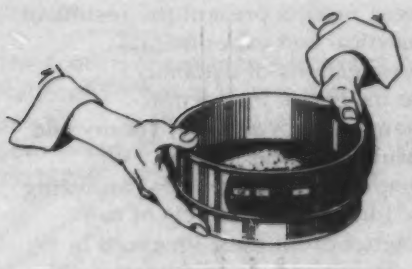
Highway Advisory Radio Operational Site Survey and Broadcast Equipment Guide, Report No. FHWA-RD-79-37

by FHWA Traffic Systems Division

This report summarizes highway advisory radio (HAR) operations activities, commercially available hardware, and licensing requirements in the United States. The report reviews HAR systems used in eight States by discussing HAR

system surveillance, advance signing, and broadcasting equipment. The report also catalogs commercially available HAR transmitters, modulating sources (tape recording equipment), and antenna types. The final section of the report focuses on the current Federal Communications Commission (FCC) rulemaking concerning HAR. Included in this discussion are HAR application and authorization requirements as well as HAR frequencies, conditions, limitations, and technical standards. All current FCC rulemaking for HAR is included in the appendixes.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 301379).



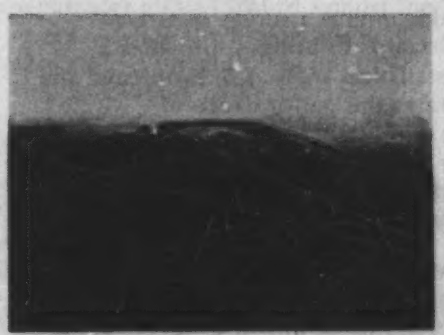
Present Aggregate Gradation Control Practices and Alternative Test Methods, Report No. FHWA-RD-79-100

by FHWA Materials Division

Aggregate gradation must be controlled in current highway construction practices to insure the production of good pavements. A survey of current control practices in the United States was conducted to discover how alternative testing methods would affect the adequacy of the control and the amount of time devoted to control practices. The results of the survey are summarized in this report.

Alternatives to the standard test methods of gradation determination were compared with the standard tests in terms of total testing time required and the accuracy of the gradation results. The sources of variation in the test results and the relative importance of each were also investigated. Recommendations on the frequencies of sampling and gradation testing are given based on the study findings.

The report is available from the Materials Division, HRS-22, Federal Highway Administration, Washington, D.C. 20590.



Investigation of the Use of Coal Refuse-Fly Ash Compositions as Highway Base Course Material: State of the Art and Optimum Use Area Determinations, Report No. FHWA-RD-78-208

by FHWA Materials Division

The need to recycle waste products is becoming more crucial as the cost of their disposal escalates and as conventional materials become scarce. Two such byproducts of the coal industry, coal mine refuse and fly ash, show promise as construction materials.

This report outlines the phase I findings of a study on the use of coal refuse-fly ash compositions as highway base course material. This portion of the investigation involved a comprehensive literature search into the engineering properties and field use of the two waste materials. An annotated bibliography was assembled and the 10 areas were identified where the use of such compositions would be optimal based on economic, environmental, and material availability considerations.

The succeeding two phases of the investigation will study the development of an optimum coal refuse-fly ash blend for each of the 10 optimum use areas based on standard laboratory tests and comparisons with conventional materials.

The report is available from the Materials Division, HRS-21, Federal Highway Administration, Washington, D.C. 20590.

Provisions for Elderly and Handicapped Pedestrians: Volume 1 (Report No. FHWA-RD-79-1), Volume 2 (Report No. FHWA-RD-79-2), and Volume 3 (Report No. FHWA-RD-79-3)

by FHWA Environmental Division

The problems and hazards experienced by elderly and

handicapped pedestrians were investigated and the results, expressed in typological form, were prioritized. These reports present research on the pedestrian environment and discuss some of the high priority environmental problems and proposals for improving accessibility. Curb ramp criteria and tactile surface materials were evaluated and the results incorporated into countermeasures that were field tested. The priority accessible network as a conceptual approach to barrier-free urban areas has been developed, and a methodology for establishing the network is described.

Volume 1, **Executive Summary,** summarizes the research results presented in Volume 2, **Hazards, Barriers, Problems, and the Law,** and Volume 3, **The Development and Evaluation of Countermeasures.**

Limited copies of the reports are available from the Environmental Division, HRS-41, Federal Highway Administration, Washington, D.C. 20590.

Countermeasures for Hydraulic Problems at Bridges: Volume I (Report No. FHWA-RD-78-162) and Volume II (Report No. FHWA-RD-78-163)

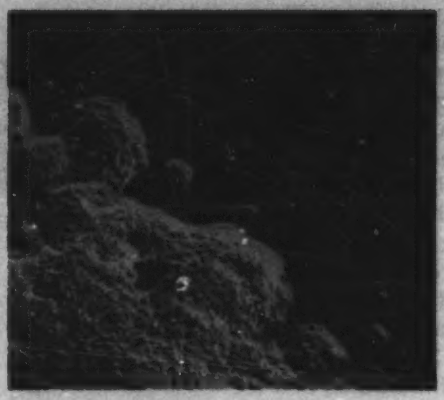
by FHWA Environmental Division

When hydraulic conditions endanger the structural integrity of a bridge, either the structure must be reconstructed or the hydraulic conditions must be improved through countermeasures. It is usually less costly to use an appropriate countermeasure such as bank revetment (riprap or slope paving), bed armoring, or flow control measures (spurs, retards, dikes, spur dikes, check dams, and jetty fields). These reports examine countermeasures applied in the United States and Canada and provide guidelines to assist design, construction, and maintenance engineers in selecting measures to reduce scour and bank erosion.

Volume I, **Analysis and Assessment,** discusses hydraulic problems, countermeasures, geomorphic factors, and bridge factors. It documents a survey of stream hazards to bridges, provides a statistical analysis of problems, and rates the performance of countermeasures. Volume I also classifies streams to account for different characteristics of stream lateral stability and behavior.

Volume II, **Case Histories for Sites 1-283,** is a collection of 224 case histories of stream sites. Each case history reports on the original bridge design, flood hazards, countermeasures used, and results.

Limited copies of the reports are available from the Environmental Division, HRS-42, Federal Highway Administration, Washington, D.C. 20590.



Investigation of Tire-Pavement Interaction During Maneuvering, Volume I (Report No. FHWA-RD-78-72), Volume II (Report No. FHWA-RD-78-102), and Volume III (Report No. FHWA-RD-78-103)

by FHWA Structures and Applied Mechanics Division

These reports present the results of analytical and experimental investigations of dynamic tire-pavement interaction phenomena. Volume I, **Theory and Results,** includes a new computerized method for analyzing the fundamental effects of tire material properties, idealized pavement surface texture parameters, sliding speeds, and contact pressures on highway vehicle tire-pavement friction. A finite element tire structure model incorporates the effects of tire geometry and material in the calculation of the shear force generated between tire and pavement—an important element in the precise evaluation of vehicle handling performance. The tire-pavement friction analysis includes 81 individual contact points. Several physical quantities including displacement, sliding velocity, shear force, and the friction coefficient are calculated at each contact point. The actual continuous distribution of each quantity is believed to be

adequately represented. Every effort has been made to realistically model tire geometry and material, although the analysis does not yet account for the effect of tread profile.

The pressure and speed dependence of the friction coefficient for low-speed sliding on the lightly wetted asphalt surfaces used in the experimental investigation were also predicted accurately.

The report recommends that further experimental friction data be gathered using actual tire tread and pavement surface specimens to investigate the range of validity of the tire-pavement friction analysis and to guide further generalization of theory.

Volume II, **Computer Program Manual,** and Volume III, **Users Manual,** further document the project.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 291257, PB 291258, and PB 291259).

Effect of Cargo Shifting on Vehicle Handling, Report No. FHWA-RD-78-76

by FHWA Structures and Applied Mechanics Division

Cargo shifting is a potential problem in practically all types of highway truck handling operations. This report describes a test program that determines to what extent shifting cargoes affect driver control of heavy tractor-semitrailer vehicle combinations. The shifting cargoes studied included water carried in low density and high density semitrailer tankers and free-hanging beef halves carried in a semitrailer refrigerator van. Vehicle handling performance was evaluated relative to baseline tests with a tractor-semitrailer carrying nonshifting cargo.

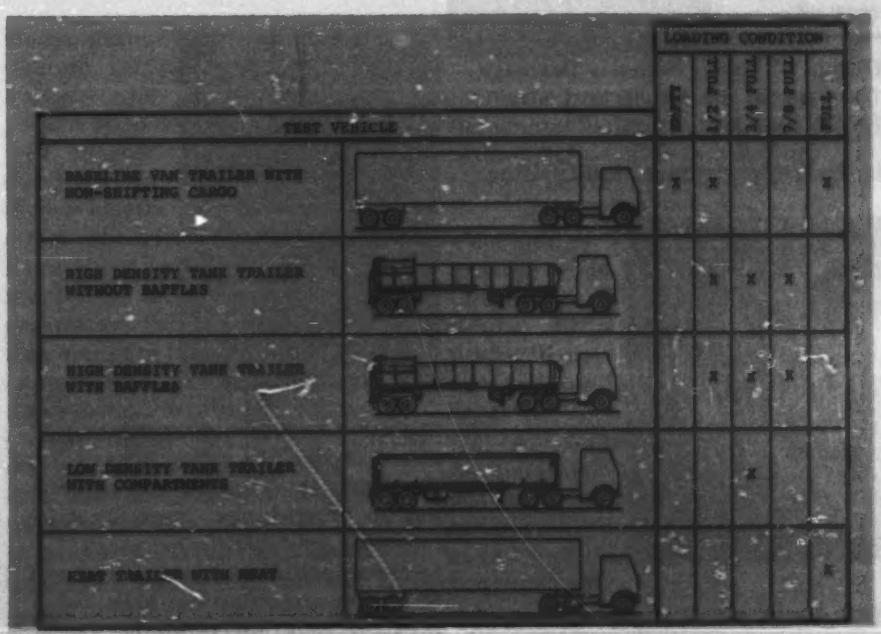
The test program included realistic vehicle braking, cornering, and lane-changing maneuvers by experienced drivers at various speeds and under various roadway geometric and pavement surface conditions. The tests were conducted with vehicle gross load levels up to 338 kN (76,000 lbf). Onboard vehicle instrumentation recorded driver inputs to steering and braking controls and corresponding vehicle dynamic responses during test maneuvers.

Films taken with a motion picture camera during braking tests verified that hanging beef halves in the semitrailer move forward against the restraint when the brakes are first applied and remain there until the stop is completed. A resultant longitudinal surge sensed by the driver under these conditions was also experienced in braking tests with the tankers.

The performance of the test vehicles during cornering maneuvers indicated that the lateral stability was principally affected by the height of the vehicle center of gravity (C.G.) The C.G., which initially is relatively high, may increase while the vehicle is in motion because of dynamic load shifts. The refrigerator van carrying hanging beef halves had a static C.G. height approximately 25 percent higher than the other test vehicles and therefore exhibited more severe lateral instability characteristics.

During lane-changing tests, large roll angles were measured in all vehicles with shifting cargoes. High density tankers had roll angles about twice as large as those of the baseline vehicle; the low density tanker and refrigerator van roll angles were about three times as large as those of the baseline vehicle.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 298110).



Implementation/User Items "how-to-do-it"

The following are brief descriptions of selected items that have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies.

U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division (HDV-20)
Washington, D.C. 20590

Items available from the Implementation Division can be obtained by including a self-addressed mailing label with the request.

Design of Urban Highway Drainage, The State of the Art, Report No. FHWA-TS-79-225

by FHWA Implementation Division

This report presents methods and procedures commonly used in the United States for urban highway stormwater drainage design. The principles, objectives, and design considerations in the current approaches to stormwater drainage include collection, storage treatment, and disposal. Each of these concepts is an integral and interrelated part of any stormwater management system, and each concept is thoroughly discussed in this report.

Limited copies of the report are available from the Implementation Division.

Traffic Stripe Removal, Report No. FHWA-TS-79-227

by FHWA Implementation Division

In the past several years much attention has been directed toward the acceptable removal of obsolete pavement markings. There are many methods available for marking removal including chemical paint removers, sandblasting, high pressure water jet, grinding, hydroblasting, and high temperature burning. Although most of these methods are successful to some degree, no one method appears to be superior for use under all conditions. This report summarizes the available methods and includes a brief description and a discussion of the effectiveness, problems, and limitations of each method. Average removal costs are also included.

Limited copies of the report are available from the Implementation Division.

Railroad-Highway Grade Crossing Surfaces, Implementation Package 79-8

by FHWA Implementation Division and the Office of Engineering

Railroad-highway grade crossings and the physical characteristics of the crossing surface are a concern to both railroad and highway authorities. A crossing requires the normal railroad track structure to be modified, which increases both construction and maintenance costs. In addition, railroad crossings disrupt normal highway surfaces and

reduce riding quality. This discontinuity in surface may result in increased vehicle operating costs and significant hazard and inconvenience to vehicular traffic.

This report presents information on available types of railroad-highway grade crossing surfaces. The information should aid in the selection and installation of suitable types of surfacing material. This report supersedes FHPM 6-6-2-3, Railroad-Highway Grade Crossing Surfaces.

The report is available from the Office of Engineering, HNG-14, Federal Highway Administration, Washington, D.C. 20590.

Hydrology for Transportation Engineers, Implementation Package 80-1

by FHWA Implementation Division

The selection and sizing of drainage structures are essential elements in the development of safe and efficient transportation systems. In the event of flooding, extensive damage may result from inadequate drainage design. Hydrologic analysis is the critical first step in drainage design. The output of this analysis is the hydraulic loading used by drainage engineers to select and size drainage structures.

This manual provides current information to assist design engineers in hydrologic analysis. It covers the essential principles, procedures, and analytical methods needed in making hydrologic studies.

The manual may be purchased for $12 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-002-00108-6).

Freeway Modifications to Increase Traffic Flow, Report No. FHWA-TS-80-203

by FHWA Implementation Division

Transportation agencies develop, operate, and maintain urban transportation systems needed to sustain economic development of an area. Because of rising travel demands and costs, buses and other high occupancy vehicles must effectively operate on the streets and freeways of an urban area. To accomplish this, techniques and procedures that provide priority treatment are being designed and implemented. The objectives of these priority treatments are to increase the people-carrying capacity of the roadway and to reduce vehicle-kilometres (vehicle-miles) of travel by encouraging a shift in mode of travel. This report addresses different roadway cross section modifications that can be made to improve the efficiency of freeways.

The report also discusses the experiences of ongoing projects. Low cost, easily implemented modifications to freeway traffic patterns, operational and safety experience resulting from lane adjustments, and suggestions for a public acceptance survey are described.

The report may be purchased for $4 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-003-00371-9).

Design of Urban Streets, Report No. FHWA-TS-80-204

by FHWA Implementation Division

Increases in the urban population in the United States have created serious deficiencies in many urban transportation services. Because of this, increased emphasis must be placed on effective operation of the existing urban street network through specialized training in design. This report provides practical, state-of-the-art information to aid in the design and operation of urban facilities and streets. Major emphasis is placed on the functional or operational safety aspects of design that apply to minor design revisions of existing facilities as well as to major reconstruction or new construction.

The report may be purchased for $10 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-003-00373-5).

Guidelines for Selecting a Cost-Effective Small Highway Sign Support System, Implementation Package 79-7

by FHWA Implementation Division

The probability of severe accidents from impacts with small sign supports is increasing because of greater numbers of small automobiles on the highways. Support systems are being reexamined to evaluate the hazards the supports present and the replacement costs as a result of accidents, vandalism, and normal wear.

This report provides criteria for selecting the most cost-effective systems from among those that can respond favorably under impact with a 1.02 Mg (2,250 lb) vehicle.

A 30-minute film, "Small Sign Supports," is also available. It describes the scope of small sign use within highway jurisdictions, the crash tests that identified the support types that responded satisfactorily to small automobile impacts, and the procedure to consider in making a cost effectiveness analysis.

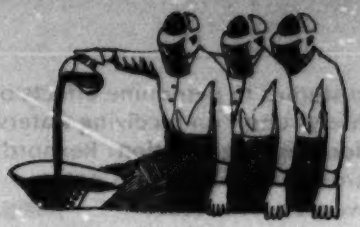
Limited copies of the report are available from the Implementation Division. The report also may be purchased for $3.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00158-6). The film is available from the Implementation Division.

New Publication

1979 Standard Highway Signs contains detailed drawings of the standard highway signs prescribed in the 1978 edition of the Manual on Uniform Traffic Control Devices. The drawings are provided to promote nationally uniform highway sign design in accordance with Title 23, U.S. Code, Sections 109(b), 109(d), and 402(a) and Highway Safety Program Standard 13, Traffic Engineering Services. The standards apply to signs used on all streets and highways, regardless of type or class or the government agency having jurisdiction. This booklet also contains general design guidelines for developing signs that conform with the basic standards. Most guide signs must be designed separately because of the variability of message or legend. An example design of an overhead advance guide sign is included. Specific design criteria are included for designing diagrammatic signs.

The booklet may be purchased for $10 from the Superintendent of Documents, U.S. Government Printing Office (GPO), Washington, D.C. 20402. There is a 25 percent discount for orders of 100 or more shipped to one address. The cost includes the design details, a looseleaf binder, and a subscription service maintained by GPO for subsequent page changes.

New Research in Progress



The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. Space limitation precludes publishing a complete list. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway or Transportation Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1M: Rural Highway Operational Safety Improvements

Title: Quantification of Two-Lane Rural Highway Problems. (FCP No. 31M1012)
Objective: Consider the design characteristics, traffic demands, traffic mix, peak volumes, traffic control, traffic conflicts, and accident characteristics as a function of the purpose, use, and class of highways.
Performing Organization: JHK and Associates, Alexandria, Va. 22304
Expected Completion Date: July 1981
Estimated Cost: $427,000 (FHWA Administrative Contract)

Title: Safety and Operational Impact of 3R-Type Projects. (FCP No. 31M1022)
Objective: Evaluate contiguous geometric and safety improvements and the relative cost effectiveness of 3R-type applications for obtaining two-lane rural highway safety improvements.
Performing Organization: Systems and Applied Sciences Corporation, Riverdale, Md. 20840
Expected Completion Date: November 1981
Estimated Cost: $291,000 (FHWA Administrative Contract)

Title: Development of Warrants for Special Turning Lanes at Rural Nonsignalized Intersections. (FCP No. 41M2542)
Objective: Evaluate the effectiveness of special turning lanes in reducing accidents and congestion, assess the cost effectiveness of these lanes, and develop warrants for their implementation.
Performing Organization: University of Nebraska, Lincoln, Nebr. 68588
Funding Agency: Nebraska Department of Roads
Expected Completion Date: June 1981
Estimated Cost: $408,000 (HP&R)

FCP Project 1T: Advanced Vehicle Protection Systems

Title: Development of a Lightweight Truck-Mounted Crash Attenuator (TMA). (FCP No. 41T6103)
Objective: Develop a TMA that is lighter, less expensive, and more effective than current designs.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95805
Expected Completion Date: June 1981
Estimated Cost: $149,000 (HP&R)

FCP Project 1Z: Implementation of Previously Completed Safety Projects

Title: Process for Safety Improvement. (FCP No. 31Z1333)
Objective: Develop a users manual and training course depicting the various processes of a highway safety improvement program (FHPM 8-2-3).
Performing Organization: Goodell-Grivas, Inc., Southfield, Mich. 48075
Expected Completion Date: January 1981
Estimated Cost: $89,000 (FHWA Administrative Contract)

FCP Category 2—Reduction of Traffic Congestion, and Improved Operational Efficiency

FCP Project 2M: Arterial Flow and Control

Title: Urban Signalized Intersection Capacity (3-28(2)). (FCP No. 52M1057)
Objective: Define terms, concepts, and computational procedures for the analysis of traffic signal controlled (pretimed and actuated) intersections for determining their level of service and capacity.
Performing Organization: JHK and Associates, Tucson, Ariz. 85701
Expected Completion Date: September 1981
Estimated Cost: $290,000 (NCHRP)

FCP Category 3—Environmental Considerations in Highway Design, Location, Construction, and Operation

FCP Project 3E: Reduction of Environmental Hazards to Water Resources Due to the Highway System

Title: Effects of Highway Runoff on Receiving Waters. (FCP No. 33E3222)
Objective: Determine the impact of highway runoff on receiving water quality and aquatic biota. Formulate guidelines for assessing impacts. Provide guidelines for conducting

field studies to determine effects of highway runoff on receiving waters.
Performing Organization: Rexnord, Environmental Research Center, Milwaukee, Wis. 53201
Expected Completion Date: October 1982
Estimated Cost: $529,000 (FHWA Administrative Contract)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Performance, Reliability, and Durability of State-of-the-Art Weigh-in-Motion Systems for Highway Vehicles. (FCP No. 45D3214)
Objective: Evaluate system performance, reliability, and durability of present state-of-the-art WIM weigh bridges and systems for acquisition of highway statistical traffic data (Phase A) and for truck overweight screening (Phase 3).
Performing Organization: California Department of Transportation, Sacramento, Calif. 95805
Expected Completion Date: June 1981
Estimated Cost: $208,000 (HP&R)

FCP Project 5E: Premium Pavements for "Zero Maintenance"

Title: Environmental Effects for Zero Maintenance Pavements. (FCP No. 35E3012)
Objective: Identify and define interdependency relationships and interaction criteria between aging, environmental, and load effects upon life cycle of pavements and their support systems. Develop techniques to evaluate interdependency and provide documentation to advance present technology leading to a complete set of procedures to account for aging, environmental, and load effects in pavements.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Expected Completion Date: December 1981
Estimated Cost: $230,000 (FHWA Administrative Contract)

FCP Project 5K: New Bridge Design Concepts

Title: Signal Enhancement Interpretation for Detection of Flaws in Reinforcing Steel and Prestressed Concrete Bridge Members. (FCP No. 35K1072)
Objective: Upgrade existing prototype FHWA magnetic field disturbance (MFD) system to incorporate improved signature acquisition and processing capability and to provide immediate readout of physical condition of reinforcing steel.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: January 1981
Estimated Cost: $126,000 (FHWA Administrative Contract)

FCP Project 5L: Safe Life Design for Bridges

Title: Acoustic Emission Monitoring of Steel Structures. (FCP No. 45L2001)
Objective: Perform acoustic emission monitoring of electroslag butt weldments in Dunbar Bridge, Charleston, W. Va.
Performing Organization: Dunegan/Endevco Corporation, Charleston, W. Va. 25305
Expected Completion Date: October 1981
Estimated Cost: $225,000 (HP&R)

FCP Project 5M: Rehabilitation and Maintenance of Low-Volume Roads

Title: Design and Construction of Low-Water Stream Crossings. (FCP No. 35M1852)
Objective: Produce guidelines with documentation for selection, design, and construction of low-water stream crossings on low-volume roads. Base the guide on a review of the state of the art and case studies to develop design criteria and methodology, system drawings, and construction procedures for use by county and local engineers.
Performing Organization: Williams and Sheladia, Mt. Rainer, Md. 20822
Expected Completion Date: October 1981
Estimated Cost: $251,000 (FHWA Administrative Contract)

Title: Interim Guidelines for Using Marginal Soil and Aggregate Materials and Waste Materials for Constructing and Maintaining Low-Volume Roads. (FCP No. 35M3013)
Objective: Identify available sources of low-cost materials suitable for use in constructing and maintaining low-volume roads. Provide guidelines for the proper use and treatment of these materials.
Performing Organization: Globetrotters Engineering Corporation, Chicago, Ill. 60605
Expected Completion Date: May 1981
Estimated Cost: $117,000 (FHWA Administrative Contract)

FCP Category 0—Other New Studies

Title: Impact of Variations in Material Properties on Asphalt Pavement Life. (FCP No. 40M1712)
Objective: Identify causes of performance problems in asphalt pavements, estimate effects on pavement life of noncompliance with existing pavements, and develop improved guidelines for controlling materials and construction.
Performing Organization: Oregon State University, Corvallis, Oreg. 97330
Funding Agency: Oregon Department of Transportation
Expected Completion Date: June 1981
Estimated Cost: $65,000 (HP&R)

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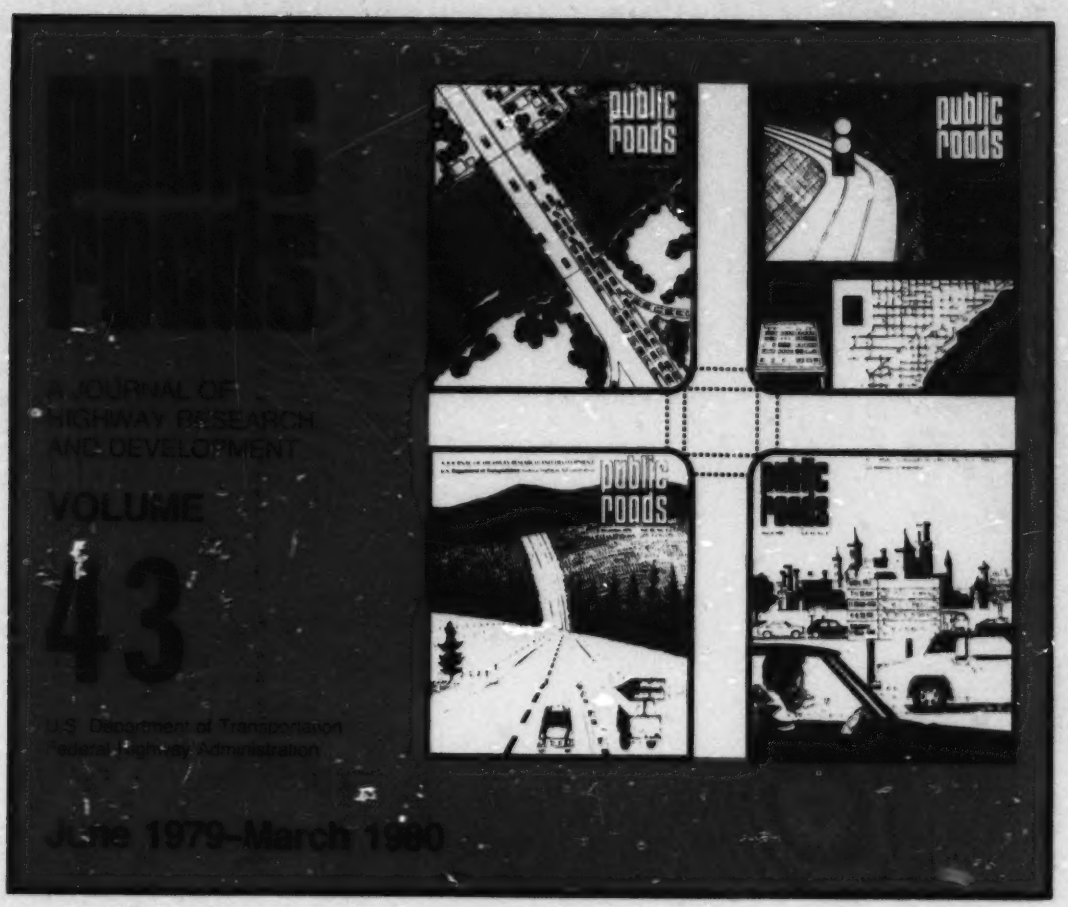

The title sheet for volume 43, June 1979–March 1980, of Public Roads, A Journal of Highway Research and Development, is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by sending a request to the editor of the magazine, U.S. Department of Transportation, Federal Highway Administration, HDV-14, Washington, D.C. 20590.

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Problems Associated With Power Failures at Signalized Intersections

Tolerable Bridge Movements Introduction

Optimizing the Expenditure of Maintenance Resources

Capacity and Quality of Flow on Urban Arterials

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September 1980 Vol. 44, No. 2

COVER:

Artist's concept of a bridge spanning a highway.

U.S. Department of Transportation
Neil Goldschmidt, *Secretary*

Federal Highway Administration
John S. Hassell, Jr., *Administrator*

U.S. Department of Transportation
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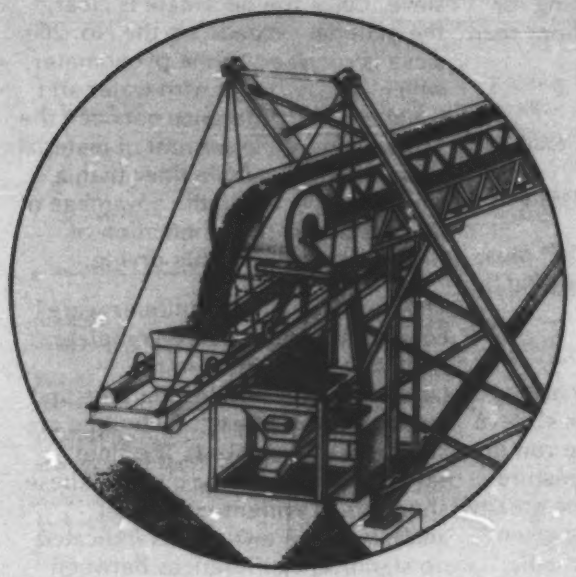

Process Control for Aggregate Production and Use

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by

Stephen W. Forster

Introduction

Process control is the maintenance or control of a certain characteristic or set of characteristics for a product during its manufacture or processing. In aggregate production, one of the characteristics controlled is the gradation of the particles, that is, the percentage of the sample retained on or passing different-sized sieves. The percentage may vary within a limited range specified for each sieve. For example, 25 to 35 percent of the sample may pass the No. 4 sieve.[1]

Gradation control insures that products made from the aggregate are acceptable and uniform. Undue changes in aggregate gradation affect, for example, the optimum moisture content and maximum density of base courses, the cement and water requirements in portland cement concrete, and the amount of binder required in bituminous mixes.

State highway and transportation departments run more than 1 million gradation process control tests annually, at a minimum of $15 per test. Because these tests require significant labor time, speeding up or automating the testing process would be cost effective. A task was established under the Federally Coordinated Program (FCP) of Highway Research and Development to examine current test methods and equipment and develop faster, more economical methods for monitoring the gradation of aggregates during production or use.

The objective of the FCP Task 4F7, "Process Control for Aggregate Production and Use," is being pursued in three ways:

- Shortcuts and alternatives to standard sieve analysis procedures are being examined. Alternative tests currently in limited use are simpler and faster than standard methods, even though these tests still are based on the sieving technique.
- Automated gradation testing is also being studied. Techniques being evaluated are not limited to variations of mechanical screening but include any technology that can be adapted to sizing aggregate. In one approach the testing device is online with the aggregate production process; therefore, samples do not have to be taken to a laboratory for testing. In another approach a prototype of an automated instrument for laboratory testing is being built.
- Several State highway and transportation department testing programs are being studied to determine the costs and benefits of shifting responsibility for most of the gradation testing from the State to the aggregate producers.

Current Practice

Under the FCP task, the Federal Highway Administration (FHWA) Region 10 Office of Federal Highway Projects (Vancouver, Wash.) surveyed the status of process control testing of aggregate gradation in the United States. Shortcuts or alternative methods that had the potential to reduce costs while maintaining the accuracy of test results were noted.

[1] Standard sieve numbers referred to in this article are the No. 4 sieve (4.75 mm [0.19 in]), the No. 10 sieve (2 mm [0.08 in]), and the No. 200 sieve (0.075 mm [0.003 in]). Sieve numbers will be used instead of measurements.

The state-of-the-art survey indicated that the frequency of aggregate sampling and gradation testing varies widely. For example, the frequency of base course aggregate sampling and testing varies from once per 225 Mg (250 tons) to once per 6400 Mg (7,000 tons). FHWA's Region 10 recommends one test per 910 Mg (1,000 tons) in most instances.

The survey also revealed numerous gradation specifications, even for a given maximum size aggregate. Probably most specifications were developed empirically, independent of what others were using. Although the differences among the specifications are minor, aggregate producers often need multiple stockpiles to meet several slightly different specifications. Regional (or national, where possible) specifications would simplify the jobs of both aggregate users and producers, particularly when the producers sell to more than one user.

Survey results showed that in almost every case, the State or other contracting agency is performing the aggregate control testing rather than the producer. The efficiency of this use of personnel is addressed later in this article.

After examining the alternative testing procedures reported, four techniques were chosen for comparison with standard sieving methods:

- FHWA's Region 10 field technique for speed drying.
- Gap sieving technique derived from FHWA Regions 3 (Baltimore, Md.), 6 (Fort Worth, Tex.), and 10 (Portland, Oreg.). *(1)*[1]
- Iowa's sieve analysis of coarse aggregate (quick method).

[1] Italic numbers in parentheses identify references on page 61.

- Kentucky's method of testing for the percentage of material finer than a No. 200 sieve by using a pycnometer.

In the Region 10 technique, standard procedures are modified by accelerating the drying process using an infrared heater to reach temperatures of 204° C (400° F) plus. This reduces the total testing time by approximately 5 hours.

The gap sieving technique determines the percent of material retained on or passing a critical sieve size. Because the test can be run on aggregate material in any moisture condition, drying time may be greatly reduced or eliminated. For a given aggregate source, there is usually one sieve size at which the material will drop out of the specification band, if it is going out of specification at all. Therefore, once this "critical" size is determined, it is necessary to monitor only that size to control overall gradation. The full range of sieve sizes can be tested periodically to check the gap sieving results.

In the Iowa quick method, only aggregate coarser than the No. 10 sieve is tested because all drying of the aggregate material is eliminated. The finer-sized material cannot be separated because of the moisture in the sample. In practice, the sample is washed over a No. 10 sieve, drained, and sieved in a wet condition. The advantage is that without drying the aggregate, accurate gradation results for the coarse material can be obtained quickly.

In Kentucky's method of determining material finer than a No. 200 sieve, the entire aggregate sample is put in a 3.8 l (1 gal) pycnometer jar that is then filled with water and weighed. Through a series of agitations and decantations, the fine material is suspended and poured from the jar onto a No. 10 sieve over a No. 200 sieve. Once the decantate is clear, the material retained on the No. 200 sieve is returned to the pycnometer, which is again filled with water and weighed. The difference between the two weights is the amount of material in the original sample finer than a No. 200 sieve. Again the advantage of this method is the elimination of time-consuming sample drying.

The four shortcut techniques were compared with standard complete wash test procedures in the laboratory. Specific testing time and labor involved in each technique are summarized in table 1. The time saved by using one or more of these shortcuts is evident. Statistical analysis of the test results indicated no significant differences between percentages of material passing a given sieve for the various test methods when compared with the standard. Field trials of these shortcuts are necessary to determine if they can adequately control aggregate gradation in practice. *(2)*

Automatic Size Analysis

An estimate of the savings from an automatic technique that are necessary to justify the initial investment in equipment can be derived from labor and testing times for sieving procedures (table 1). Within these economic constraints, two studies were initiated to address the automation of aggregate gradation testing. One study investigated an automated, online sampling and testing system, and the other study is developing a prototype laboratory testing instrument.

Online systems

The first study investigated available sizing techniques for their adaptability to online use at the aggregate plant. There were no constraints on the techniques studied; that is, techniques were not limited to sieving. Online systems

Table 1.—Testing times for shortcut techniques using a 16 kg (35 lb) sample

Test technique	Times										
	Initial weighing, drying, and cooling	Initial sieving, cleaning of sieves, and calculation	Batching or splitting	Washing for passing No. 200 sieve	Drying, cooling, and sieving wash sample. Cleaning sieves, and weighing wash sample	Washing material over No. 10 sieve	Placing sample in pycnometer for initial and final weighing	Draining sample	Final calculation	Total test	Actual labor
	Minutes	*Minutes*	*Minutes*	*Minutes*	*Minutes*	*Minutes*	*Minutes*	*Minutes*	*Minutes*	*Hours*	*Hours*
Standard complete wash[1 2]	149			35	259				5	7.47	2.05
Region 10's speed drying[1 2]	27	27	9	6	88				9	2.76	1.38
Gap sieving (one sieve method)[1 2]	31				9				2	0.70	0.22
Kentucky's pycnometer method	2		9	21			30		4	1.10	1.10
Iowa's quick method	2				25	9		12	5	0.88	0.88

[1] Six percent moisture before initial drying.
[2] Drying temperatures: Standard complete wash = 110° C; Region 10 = 156° C; Gap sieving = 160° C.

$$°C = \frac{°F - 32}{1.8}$$

were investigated for two reasons. First, any automation of the sampling/testing procedure decreases the labor time involved for each gradation test. This either reduces the technical staff required or frees the current staff for other work. Second, in many aggregate plants, considerable time is spent obtaining and transporting the samples from the belt, stockpile, or other source to the laboratory for testing (fig. 1). If the material could be sampled and tested automatically during production (as it came off a belt, for instance), the time-consuming and sometimes hazardous task of sample gathering would be eliminated.

In the online systems study, researchers surveyed the literature, manufacturers, patent files, and professional societies for sizing techniques currently used within and outside the aggregate industry. A panel of experts, including aggregate producers and trade association representatives, State highway and transportation department representatives, and experts in general-sizing techniques, advised the researchers throughout the study. The panel aided in developing specifications for an online sizing system that would be technically and economically practical for use in plants producing over 91 000 Mg (100,000 tons) per year. The specification developed includes plant coverage, sampling, measurement/analysis, and annual cost.

Plant coverage: To meet the plant coverage requirements, the system should be economically applicable to plants producing 90 to 1800 Mg (100 to 2,000 tons) per hour. Although many plants produce less than 90 Mg (100 tons) per hour, their impact on total production is relatively small. For instance, plants producing 23 000 Mg (25,000 tons) or less per

Figure 1.—Sampling aggregate for gradation testing.

year of aggregate constitute 36 percent of the sand and gravel operations and 40 percent of the crushed stone operations in the United States. However, these same plants produce only 3.5 percent of the total sand and gravel and 2 percent of the total crushed stone in the United States.

Sampling: Sampling should be done where the material is flowing to minimize segregation of the aggregate. Random samples should be taken according to the sample weights listed in American Society for Testing and Materials test method C 136–76. *(3)*

Measurement/analysis: Samples must be measured over a range of the No. 200 sieve to 102 mm (4 in). This range, which includes particles more than 1,000 times as large as others, posed a problem for the researchers. Further, if the measurement technique is based on particle number (as opposed to mass), the distribution obtained must be converted to a mass distribution to be comparable with sieve analysis. This means that particles smaller than the No. 200 sieve must be counted (in large numbers) to include the majority of mass passing the No. 200 sieve. In the specification, allowable measurement error is equivalent to that found in present sieve analysis methods from variation in both the sieve openings and sieve fraction weights. The specification also states that a result should be available 20 minutes after the sample is obtained.

Annual cost: The specification sets the upper limit of the annual equipment cost at 10 percent of the plant's annual sales. This cost includes the installed purchase price of the equipment amortized over 15 years, as well as loan payments, and operation and maintenance costs. The purchase price is therefore approximately $110,000 for a plant producing 91 000 Mg (100,000 tons) of aggregate per year.

Forty-five potential techniques were identified during the initial survey. Of these, the following six warranted detailed study: Optical array image analysis, hydrocyclone separation, air table (specific gravity) separation, trommel (cylindrical screen) separation, vidicon (television image) counting and measuring, and optical shadowing. Detailed study revealed inherent limitations in four of the techniques. As a result, the final design work focused only on the vidicon system and the optical shadowing technique.

Vidicon system

Vidicon cameras previously have been used for counting and measuring material. The vidicon system stores optical images as latent electrostatic charges on the surface of a target layer. This target layer is scanned by a low-energy electron beam that produces a video signal. The video signal then can be used to reproduce the image or record it onto other media. One advantage of a vidicon system is that it is a noncontacting, nondestructive technique. Particle sizes between 0.01 mm and 102 mm (0.0004 in and 4 in) can be detected by choosing the appropriate optical systems.

A vidicon system that analyzes particle size requires several basic components (fig. 2). The controlled light source illuminates the particles for projection onto the vidicon scanner, which minimizes noise and optimizes resolution and speed. The hardware electronics convert the signal from the scanner so that the information can be analyzed by computer and translated to an image on the video monitor.

In an aggregate plant, one data analysis subsystem could be centrally located, with a light source and vidicon camera located on each product line to be monitored. Data from the vidicons would be processed at the same data analysis subsystem to reduce the cost for multiple setups. Also, the service life of the data analysis electronic equipment would be prolonged because the equipment could be housed away from the harsh environment surrounding the production line.

Several problems inherent in any attempt to automate gradation testing were identified during the process of adapting the vidicon system to aggregate gradation measurement.

Measurement location: To prevent erroneous measurements caused by coincidence, the sensing device must be located where aggregate particles can be adequately dispersed. The best way to adequately disperse the particles is to remove some material from the main product flow and then control the flow of this sample through the sensing device.

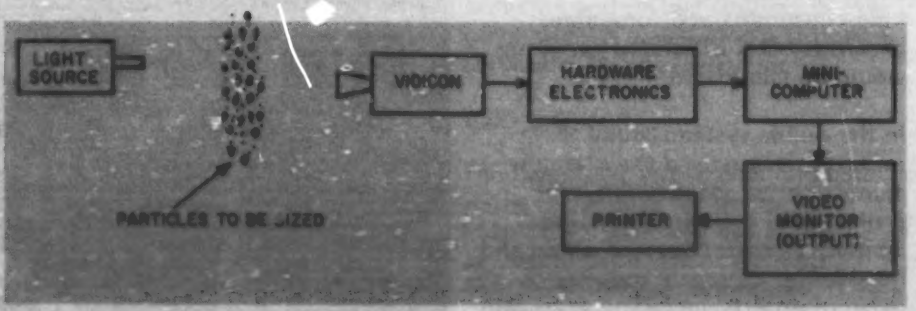

Figure 2.—Typical vidicon system arrangement.

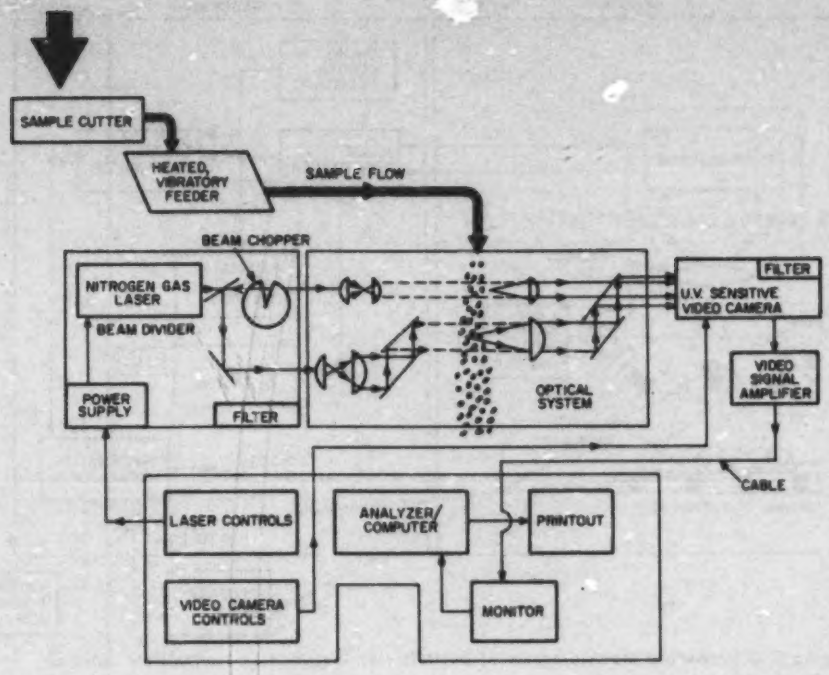

Figure 3.—Vidicon system functional schematic.

Agglomeration, occurring when small particles cling to each other or to larger ones, can be minimized by drying the aggregate and breaking down the adhesion. A heated, vibrating feeder positioned in front of the vidicon camera can accomplish this.

Installation requirements and operating environment: Continued reliability of a vidicon system in an aggregate plant requires an environmental enclosure, shock mounts, and an optical cleaning device. The video monitor and image analyzer can be located away from the production line in a more favorable environment.

A schematic diagram of the complete vidicon system and support devices necessary for sampling and measuring is shown in figure 3. Figure 4 is a conceptual drawing of how the system might appear installed on a conveyor belt line.

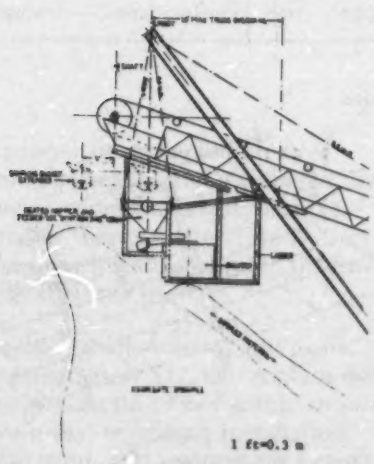

Figure 4.—Typical arrangement of vidicon system mounting at head of product conveyor.

Range of particle sizes: As previously noted, the range of particle sizes to be measured (No. 200 sieve to 102 mm [4 in]) is an obstacle with any automating technique. To successfully measure a sample with some particles more than 1,000 times as large as others, two lens systems that could time-share the vidicon camera would be needed. Each lens system would measure half of the particle-size range.

Coincidence and agglomeration: Particle measurement can be inaccurate if particles coincide or agglomerate. Coincidence occurs when small particles touch and are counted as one particle or when small particles are hidden by larger particles. Coincidence can be minimized by insuring that the aggregate particles to be measured are adequately dispersed.

The cost of a vidicon system to monitor a plant with five production lines was estimated. Based on quoted prices for the equipment already in existence and engineering and production estimates for the remaining components, such an installation was estimated at $212,000. Including operation and maintenance costs, the annual cost for operating the system is estimated at $46,000.

The estimated purchase price of the vidicon system described is approximately twice that allowed by the original specification, which probably excludes an additional number of plants from using this system. However, in applicable plants it would be a powerful process control tool, determining gradation on each of five production lines every 20 minutes.

Optical shadowing

The second technique investigated is based on optical shadowing. A particle-size distribution transmitter, a commercially available device designed for the mining and mineral industries, sizes particles on a conveyor belt by measuring the difference in reflectivity of the aggregate particles and the shadows among them (figs. 5 and 6). To maximize this difference, the material is illuminated at a low angle to insure shadowing among the particles. The sensor head is positioned above the belt and between two lamps that provide the low angle illumination. The sensor consists of a lens system, two photodetectors, and preamplifiers connected to the data processing unit. The photodetectors scan illuminated areas (particles) and shadow areas (between particles) along a narrow band in the center of the belt. The signal from these optical detectors is converted to an electric analog signal from which a threshold value is determined. Every feature with a value above the threshold is considered a particle and every feature with a value below is considered a space. A particle chord length is determined from the time it takes the particle to pass under the detector, the speed of the belt, and the magnification of the lens system.

Certain characteristics of the device make it more suitable for controlling the aggregate process than for determining gradation. Measurements are made only along a narrow band down the center of the belt. Thus, if any material segregates, the sample will not be representative. Similarly, measurements made only at the upper surface of the material probably will not be representative. Also, the device measures chord length, which may have only an indirect relationship to particle

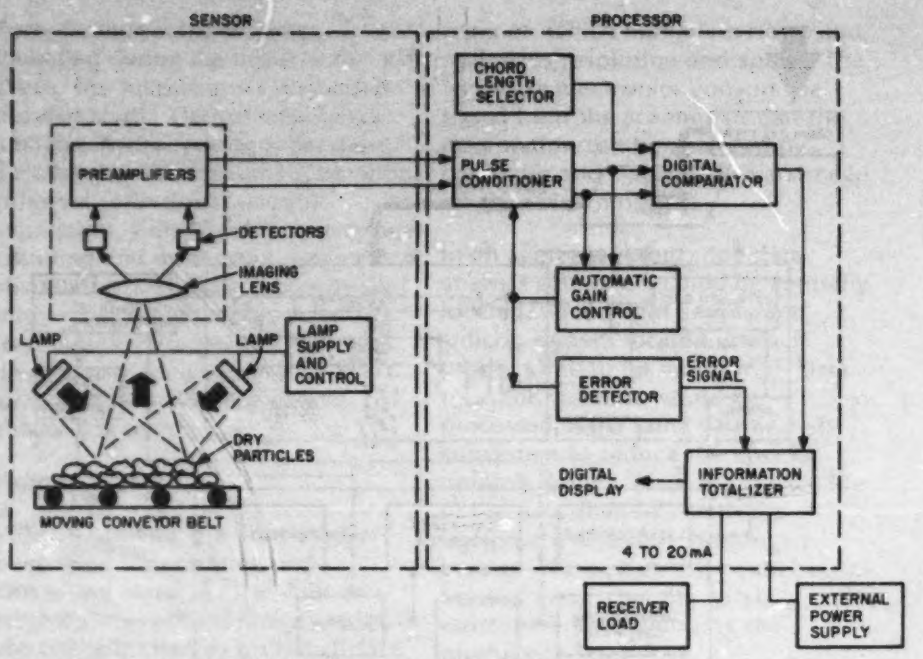

Figure 5.—Simplified block diagram of particle-size distribution transmitter (optical shadow technique).

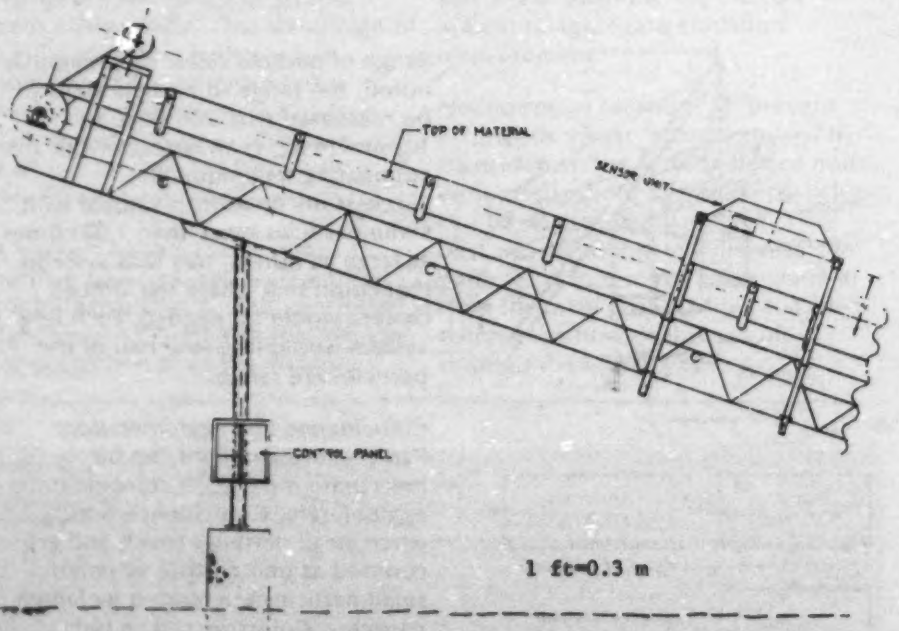

Figure 6.—Typical arrangement of optical shadow mounting on product conveyor.

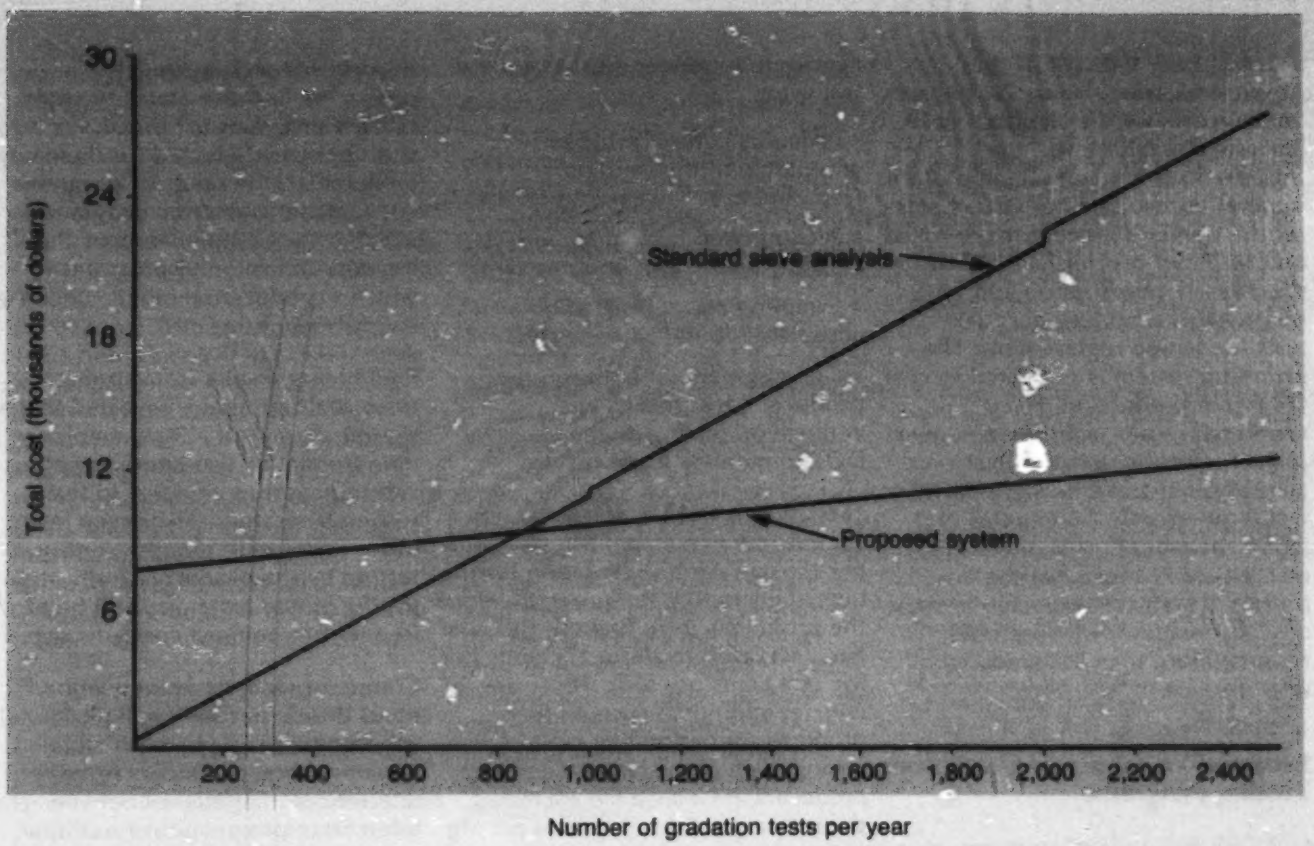

Figure 7.—Gradation testing cost comparison.

diameter or projected area for nonspherical particles. Finally, the device cannot measure particles smaller than 3 mm (0.1 in) in diameter because of poor shadowing with this size particle.

An optical shadowing unit costs approximately $12,000. A plant with five production lines would require five units. Including installation, operation, and maintenance costs, a five-unit system would cost approximately $12,500 annually. Gradation determinations for each of the five lines would be supplied every 20 minutes.

Reference 4 summarizes investigations of online systems and includes complete descriptions of the most promising techniques.

Automated laboratory testing instrument

A second approach to automatic size analysis for gradation control was to develop a portable laboratory instrument that would reduce both labor and testing times. Because the system would not be tied to the production line, it would be adaptable to various situations. For example, State highway and transportation departments could use it to test samples in their central or district laboratories if the quantity of aggregate samples was sufficient to justify using the instrument.

As in the study of online systems, the problem of accurately measuring a wide range (material finer than the No. 200 sieve to 102 mm [4 in]) of particle sizes was evident. Extensive study indicated that two subsystems were necessary—one for material larger than the No. 4 sieve and one for material smaller than the No. 4 sieve. Prototypes for each of the subsystems are being constructed for FHWA and are expected late in 1981. Both subsystems will pass the aggregate particles between a laser and photodetector arrays. The particles will block some of the light from the arrays and thereby can be measured and counted. Hardware for the two subsystems will differ significantly because of the different range of particle sizes to be measured.

Preliminary cost analyses have determined the number of tests that a laboratory would have to run per

year to justify the cost of the proposed system. Figure 7 compares annual costs for the standard sieve analysis procedure with the proposed system as a function of the number of tests run per year. Costs for the standard procedure are based on $10.78 per test for labor, $388 for fixed yearly amortization and maintenance, and $300 per 1,000 tests for screen replacement. The proposed system is projected to cost $60,000 in production. Fixed maintenance plus amortization over 10 years is estimated at $7,800 annually; however, because each test requires only 7.5 minutes of labor, the labor cost is only $2 per test. Figure 7 shows that the proposed system is expected to show cost advantages for laboratories running more than 800 gradation tests per year.

Economic Analysis of Gradation Testing Programs

FCP Task 4F7 to develop faster, more economical methods of aggregate gradation was not limited to investigations of techniques and equipment. Research also addressed the question of whose personnel—the State highway and transportation department's or the aggregate producer's—can perform process control sampling and testing the most efficiently and economically. State highway and transportation departments often assign an employee to an aggregate plant to test material used in State construction work. The efficiency of this procedure is questionable. An employee of the producer (who is already at the site) could test the material and thus free the State employee for other duties.

Georgia has instituted voluntary producer certification so that producers can maintain their own process control. *(5)* As part of the certification requirements, an aggregate producer must attain the following:

- A quality control program acceptable to the Georgia Department of Transportation (DOT).
- An approved laboratory with certified equipment and personnel.
- Sampling and testing frequencies acceptable to the Georgia DOT.
- Good agreement between the producer's test results and verification testing by the Georgia DOT to monitor these results.

The Georgia DOT estimated it was spending 4.1 cents per Mg (4.5 cents per ton) on control gradation testing before instituting the program. Since the program's introduction, this cost has decreased to about 1.3 cents per Mg (1.4 cents per ton). There are, of course, additional costs to the producer for performing more of the testing. One of the largest aggregate producers in Georgia has estimated this increase at 0.9 to 1.4 cents per Mg (1.0 to 1.5 cents per ton). *(5)* Overall, then, the program is saving 1.4 to 1.9 cents per Mg (1.6 to 2.1 cents per ton) of aggregate. Based on an annual aggregate production of approximately 36 million Mg (40 million tons), the total cost reduction is $400,000 per year.

There are other benefits from the program. Producers have greater flexibility in plant operations because material can be produced and shipped any time in response to consumer needs rather than only when State inspectors are available. Also, because the producer is doing the control testing, a testing schedule that gives the producer optimum control of the product without overtesting can be developed.

A study was undertaken to seek additional information on the most efficient process control program. Onsite visits were made to selected State highway and transportation departments and aggregate producers in those States to study current programs for process control testing. States selected for the study represented the range of programs currently in use—from programs in which a State employee does the majority of testing to programs in which a producer employee does the majority of testing.

Preliminary results from studies of three State programs revealed that a significant part of a State employee's time frequently was spent sampling and transporting samples to the testing laboratory. Also, even in those States with process control testing by the producer, verification testing by the State may still be too frequent to be most cost effective.

Comparisons of combined labor times (hours per 910 Mg [1,000 tons]) devoted to testing by both State personnel and producers reveal differences in efficiency between the three State programs. For example, both States A and B have transferred much of the process control testing to the producer. State A personnel spend 15.4 hours per 910 Mg (1,000 tons) for gradation testing. The aggregate producers in State A spend approximately 2.3 hours per 910 Mg (1,000 tons), bringing the total gradation testing effort to 17.7 hours per 910 Mg (1,000 tons).

State B personnel, on the other hand, devote 2.8 hours per 910 Mg (1,000 tons) to gradation testing; aggregate producers spend 0.8 hours. The total is thus only 3.6 hours spent on gradation testing for every 910 Mg (1,000 tons) of aggregate.

The difference in labor times between State A and State B appears to be in the amount of verification testing performed by the States. The amount and frequency of verification testing by the States should be

examined in relation to program efficiency.

Unlike States A and B, State C personnel perform all gradation testing. The State devotes 140.8 hours to gradation testing per 910 Mg (1,000 tons), while producers average only 0.3 hours per 910 Mg (1,000 tons). The combined total is 141.1 hours per 910 Mg (1,000 tons) for gradation testing. A significant portion of the total State time is spent in travel—collecting samples and taking them to the laboratory. However, travel is an integral part of this kind of program and therefore must be included in any comparisons. The excessive time devoted to gradation testing in this traditional program is evident.

The final report on State process control programs is expected in the fall. The report will discuss the data collected and will include a recommended program for the States based on economic analyses of the various programs investigated.

Summary

Previous research indicates that gradation control during aggregate production is necessary for satisfactory products. The research described in this article was performed to develop less expensive and faster methods of aggregate gradation control testing.

In a survey of current State practices, alternatives to time-consuming standard sieve analysis procedures were evaluated in the laboratory. The alternative procedures included methods to either reduce or eliminate drying times or reduce the number of sieves required. Although the procedures appear to control gradation, field evaluation is necessary. If the procedures are adequate, they could be implemented easily because specialized equipment is not required.

To reduce the high labor costs of gradation testing, efforts are being made to automate the procedure. Attempts have been made to automate the entire sampling and testing process using a system online with the aggregate production. In addition, a laboratory instrument is being designed and built for automating just the gradation test. Further development of the online equipment by FHWA will depend on developmental results of the prototype laboratory instrument.

Various State programs for process control also are being analyzed. The question being addressed is whether it is more economical for State employees or producer employees to do the majority of the process control testing. Preliminary results indicate that testing can be done more efficiently and economically by the producer (with limited verification testing by the State) without losing product quality.

Field trials will be performed for the four shortcut testing techniques presented and the prototype laboratory instrument. These field trials will evaluate how well these approaches work in daily operations.

A final report of the FCP Task 4F7 research will discuss the approaches examined during the task, the conclusions reached, and the benefits from the various approaches. The report will include results of the field trials as well as a discussion of the implications of the economic analysis. States will be encouraged to apply the various approaches through the Highway Planning and Research Program.

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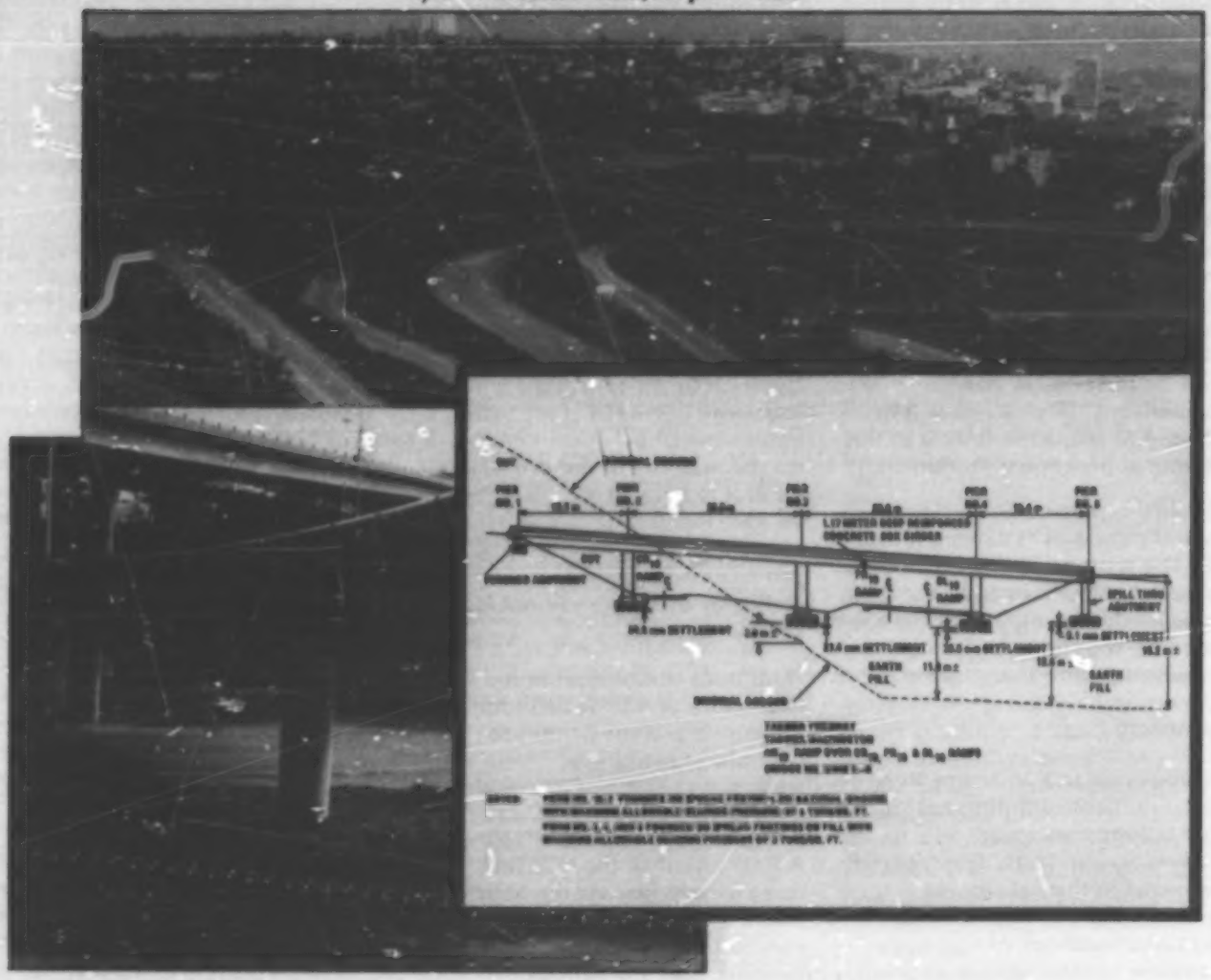
[3] Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

Stephen W. Forster is a geologist in the Materials Division, Office of Research, Federal Highway Administration. He is task manager for Task 4F7, "Process Control for Aggregate Production and Use," in the Federally Coordinated Program of Highway Research and Development. Since joining FHWA in 1975, Dr. Forster has worked in the areas of rapid testing of aggregate gradation, skid resistant aggregates, and more durable aggregates for pavements.

Bridge Movements and Their Effects

by
Lyle K. Moulton and Joseph R. Kula



This is the first in a series of three articles on tolerable bridge movements. The series was introduced in the last issue of *Public Roads*, vol. 44, No. 1, June 1980. This article discusses the collection and analysis of field data on bridge movements and the ability of bridges to tolerate these movements. It was concluded that many bridges, depending upon kind of span, length of span, and kind of construction material, can tolerate significant levels of differential settlement without sustaining intolerable structural damage. However, the horizontal movement of substructure elements was found to be more critical than vertical movement and must be controlled closely to avoid structural damage.

Introduction

A structure type that considers functional, architectural, engineering, and economic aspects usually is selected before buildings, bridges, and other structures are designed. A preliminary design is prepared for the geotechnical engineer with a request for recommendations for founding the structure. Subsurface explorations, sampling, and geotechnical analyses are conducted, in addition to bearing capacity evaluations and estimates of immediate and long term total and differential movements. The estimates are used to decide how the structure should be founded to be safe and

economical. Often, one of the major considerations is whether or not the structure can tolerate the estimated total and differential movements.

Much data have been collected and used to establish criteria for tolerable movements of buildings and some industrial structures. *(1–5)*[1] Unfortunately, adequate criteria are not presently available for highway bridges, so foundation selection for bridges is often based on conservative rules of thumb or textbook guidelines, building codes, and specifications. Current American Association of State Highway and Transportation Officials (AASHTO) specifications state: "In general, piling shall be used when footings cannot, at a reasonable expense, be founded on rock or other solid material." *(6)* However, using arbitrary or restrictive guidelines for the tolerable movements of bridges often has led to the unwarranted and uneconomical use of piling or other deep foundations. Under certain circumstances, bridges founded on piles or other deep foundations can move—sometimes substantially. *(7–13)*

To satisfy the need for more rational criteria for the tolerable movements of highway bridges, the Federal Highway Administration (FHWA) set out to develop bridge design criteria that recognize the interaction between the foundation and the bridge structure and establish tolerable bridge movement limits for a range of bridge types. Although this research was divided into several tasks and subtasks, the following three general study categories evolved:

- A state-of-the-art assessment of tolerable bridge movements based on a literature review, an appraisal of existing design specifications, and a compilation and analysis of movement, damage, and tolerance data from bridges in the United States and Canada.
- A series of analytical studies to determine the limits of tolerable foundation movements, with respect to structural integrity and serviceability, for various concrete and steel bridge types.
- A study to develop methodology for optimizing the design of superstructure and substructure systems as an integrated unit, and recommendations for modifying current design specifications.

This article summarizes the first of these studies. The remaining two studies will be discussed in subsequent articles.

[1]Italic numbers in parentheses identify references on page 75.

Collection and Analysis of Field Data

Sources of data

In 1967 and 1975, Transportation Research Board (TRB) committees[2] sent questionnaires to highway agencies throughout the United States and Canada to survey bridge movements. *(14–17)* In addition to identification information, the questionnaires requested information on the year the bridge was completed, the kind and number of spans, the kind of abutment, soil and foundation conditions, and estimated or observed movements and their effects on the structure. Some of the highway agencies supplied soils reports and design drawings as well. Overall, information was supplied by 34 States, the District of Columbia, and 4 Canadian provinces.

As a first step in the current investigation, various highway agencies were asked to supply additional information, including boring logs, settlement data, as-built plans, and tolerance ratings, for those bridges that had been included in the 1967 and 1975 surveys. Information was also requested on any bridges not included in the 1967 and 1975 survey responses that had experienced movement. Supplementary data were supplied by 15 States and from an FHWA study on 28 bridges in the State of Washington.[3]

Data are now available on 200 bridges that have experienced some movement, and as-built plans have been obtained for 94 of these bridges. At present, the data from 180 bridges have been analyzed.

Assumptions and definitions

The data have certain limitations. Because a substantial amount of the data was obtained by questionnaires, the quality of the data depends on the information requested in the questionnaire and the completeness and accuracy of the responses. In some instances the responses were incomplete or unclear, and there was a general lack of

[2]The 1967 survey was conducted by TRB Committee SGF-B3 and the 1975 survey was conducted by TRB Committee A2K03.

[3]"An Evaluation of the Performance and Cost Effectiveness of the Use of Spread Footings in Fill Embankments for the Support of Highway Bridge Abutments in the State of Washington," thesis by S. J. Seguirant for Master of Science in Civil Engineering, University of Washington, Seattle, Wash., 1979. "A Field Evaluation of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill," Report No. FHWA/RD-80/044, *Federal Highway Administration*, Washington, D.C. Not yet published.

common terminology. Consequently, definitions and simplifying assumptions were adopted to generalize the data for classification and analysis:[4]

Vertical displacement—When applied to structural damage, this term includes the raising or lowering of the superstructure above or below planned grade or a sag or heave in the deck. Structures requiring shimming or jacking as well as truss structures with increased camber are also included.

Horizontal displacement—When applied to structural damage, this term includes the misalinement of bearings and the superstructure or beams jammed against the abutments. Also included in this category are bridges whose superstructure extended beyond the abutment or whose beams required cutting, bridges having an open space between the abutment or pier seats and the beams, and bridges experiencing horizontal movement of the floor system.

Tolerable—The subjectivity of this term is one reason for the lack of generally accepted tolerable movement criteria. The TRB committee defined intolerable movement as follows: "Movement is *not* tolerable if damage requires costly maintenance and/or repairs *and* a more expensive construction to avoid this would have been preferable." *(16)* For the sake of consistency, this definition was adopted for the FHWA study.

Data analysis methods

The field data were analyzed to delineate trends of bridge foundation movements, their effects, and the ability of the bridge to tolerate these movements. Three separate analyses were conducted, each with a different methodology.

The first analysis involved investigating the influence of substructure variables—general soil condition, foundation type (spread footings or piles), abutment type (full height, perched, or spill-through), and height of approach embankment—on bridge abutments. Table 1 summarizes the substructure data that were incorporated into this analysis. Combinations of these variables were considered to determine if they result in foundation movement.

Additional variables—span type (simply supported or continuous), and the abutment-embankment-pier geometry—were investigated to determine their influence on pier movements. Table 2 summarizes the superstructure data that were incorporated into this analysis. Again, the influence of each of the selected variables was considered separately and in selected combinations.

The second analysis involved investigating the influence of bridge foundation movements on the bridge structure to determine what kinds and magnitudes of movements most frequently cause structural damage. Kind of movement (vertical only, horizontal only, or vertical and horizontal), magnitude of movement (maximum differential vertical movement between two successive abutments or piers and maximum horizontal movement), span type, kind of structural material (steel or concrete), number of spans, and abutment type were considered. Table 3 summarizes the kinds of structural damage and the bridges that experienced them. Many of these bridges experienced multiple damaging effects.

Table 1.—Substructure data

	Number of bridges
General soil conditions	
Fine grained soils	62
Granular soils	17
Fine grained soils over granular soils	16
Granular soils over fine grained soils	25
Interlayered/intermixed soils	43
Bedrock	7
Permafrost soils	2
Soil conditions not given	8
Foundation type	
Spread footings	48
Piles	80
Abutments on spread footings/piers on piles	14
Abutments on piles/piers on spread footings	19
Abutments and piers on both spread footings and piles	10
Abutments on caissons/piers on spread footings	2
Foundation type not given	5
Abutment type	
Full height	36
Perched	79
Spill-through	35
Perched, spill-through	3
Abutment type not given or unknown	27
Height of approach embankments	
Metres	
Cut	2
0 to 3	11
3 to 6.1	35
6.1 to 9.1	61
9.1 to 12.2	21
12.2 to 15.2	10
15.2 to over 30.5	10
Approach heights vary	8
Approach height not given	22

1 m=3.3 ft

[4] "Analysis of Bridge Movements and Their Effects," thesis by J. R. Kula for Master of Science in Civil Engineering, West Virginia University, Morgantown, W. Va., 1979.

Table 2.—Superstructure data

Span type	Number of bridges
Simple	71
Continuous	45
Rigid frame	8
Cantilever	4
Miscellaneous or not given	52
Structural material	
Steel	73
Concrete	55
Steel and concrete	6
Material type not given	46
Number of spans	
One	20
Two	16
Three	61
Four	30
Five	12
More than five	35
Number of spans not given	6

Table 3.—Structural damage

Structural damage	Number of bridges
Abutment damage	47
Pier damage	13
Vertical displacement	24
Horizontal displacement	41
Superstructure distress	68
Damage to rails, curbs, sidewalks, parapets	18
Bearing damage	38
Poor riding quality	11
Not given or corrected during construction	8
None	26

The third analysis involved investigating the various bridge structures' tolerance to movements. Kind of structural damage, kind of movement, magnitude of movement (maximum differential vertical movement between successive units of the substructure, maximum angular distortion, and maximum horizontal movement), span type, kind of structural material, number of spans, and abutment type were considered.

Influence of Substructure Variables on Foundation Movements

Abutment movements

Three hundred thirty-nine abutments were analyzed. Table 4 summarizes data for the 248 abutments that experienced movement. These data show that most of the abutments moved vertically, one-half of them moved horizontally, and almost one-third of them simultaneously moved vertically and horizontally. The magnitudes of the vertical movements were greater than the horizontal movements. In many instances the abutments moved inward until they jammed against the beams or girders, which acted as struts and prevented further horizontal movement. Table 4 also shows that abutment movements tended to be larger and more variable for those abutments that moved both vertically and horizontally.

Table 5 summarizes abutment movements by abutment type. Because perched and spill-through abutments had vertical and horizontal movements over a wider range than did the full height abutments, foundation system analysis and design for perched and spill-through abutments needs further study. The conventional design advantages of using perched or spill-through abutments rather than full height abutments may be considerably offset by the larger probability of intolerable movements occurring.

Abutments founded on spread footings moved more than abutments founded on piles. Table 6 shows that abutments on spread footings experienced slightly larger vertical movements but smaller horizontal movements than did those founded on piles. These same trends were

Table 4.—Abutment movements

Movement	Number of abutments	Percent moved	Range	Average	Standard deviation
			Millimetres	*Millimetres*	*Millimetres*
All types	248	100			
Vertical	197[1]	79.4	2.5-1280.1	154.9	193
Horizontal	125	50.4	7.6- 457.2	83.8	81.3
Vertical and horizontal	76	30.6	10.2-1280.1 7.6- 457.2	210.8 88.9	254 94

[1] Two abutments that raised vertically are not included.

1 mm=3.9 × 10^{-2} in

Table 5.—Movements by abutment type

Abutment	Movement	Frequency: Number of abutments	Frequency: Percent moved	Range	Average	Standard deviation
				Millimetres	*Millimetres*	*Millimetres*
Full height	All types	62	100			
	Vertical	55[1]	88.7	7.6–518.2	91.4	94
	Horizontal	30	48.4	7.6–203.2	61	55.9
	Vertical and horizontal	25	40.3	12.7–518.2 7.6–203.2	109.2 66	119.4 61
Perched	All types	118	100			
	Vertical	94	79.7	2.5–965.2	170.2	208.3
	Horizontal	58	49.2	7.6–457.2	88.9	86.4
	Vertical and horizontal	34	28.8	20.3–965.2	274.3 99.1	279.4 99.1
Spill-through	All types	37	100			
	Vertical	26	70.3	12.7–609.6	200.7	193
	Horizontal	19	51.4	25.4–228.6	83.8	66
	Vertical and horizontal	8	21.6	12.7–396.2 25.4–139.7	144.8 48.3	157.5 38.1

[1] Includes two full height abutments that raised 76.2 mm vertically.

1 mm=3.9 × 10^{-2} in

observed when the data were analyzed by abutment type; however, the differences in average movements between spread footing and pile foundations were not large. This finding, coupled with the relatively large number of pile foundations that did move, tends to refute the generally accepted notion of the superiority of pile foundations in limiting abutment movements, particularly for perched abutments in fills. These findings also contradict those reported for Ohio bridges. *(14)*

Vertical movement was frequent for abutments founded on spread footings on predominantly fine grained soils. Horizontal movement occurred most often for pile foundations in fine grained soils overlying granular soils. The largest vertical movements occurred for abutments on spread footings in fine grained soils and on pile foundations in granular soils overlying fine grained soils. The largest horizontal movements occurred for pile foundations and spread footings in fine grained soils.

Approach embankment heights did not correlate significantly with the frequency and magnitude of abutment movements, which agrees with the findings for Ohio bridges. *(14)* As might be expected, frequency and magnitude of vertical movements increased with an increase in height of approach embankments. However, additional analyses of embankment height, considering abutment type, foundation type, and soil conditions, did not show meaningful trends.

Pier movements

The pier movements analysis showed that piers move less often than abutments. Only 31 percent of the 636 piers analyzed moved, compared with 73 percent of abutments. Table 7 shows that vertical movements for piers were considerably less than for abutments. Unlike the abutment movements, horizontal pier movements were larger than the vertical movements. Also, unlike abutment movements, the magnitudes of movements for piers with both vertical and horizontal movements were smaller than for piers with movement in only one direction.

Although there were more piers founded on piles than on spread footings, about half of those that moved were founded on spread footings. This suggests that founding piers on piles is more successful than founding abutments on piles, particularly for perched and spill-through abutments. Table 8 shows that the average magnitudes of movements were similar for both spread footing and pile foundations. However, the range of vertical movements was wider for the piers on spread footings than for piers on piles.

Few trends were evident on the relationship of pier movements with soils and foundation types. Movements were most frequent for spread footings on fine grained soils. A surprising finding was that over 33 percent of 95

piers on piles in granular soils moved, compared with just 17.5 percent of 206 piers on piles in fine grained soils.

Piers in or near the toe of approach embankments moved almost twice as often as piers away from the embankment. The data showed that, contrary to what might be expected, vertical movements were slightly larger for piers away from the embankment, with an average movement of 79 mm (3.1 in), compared with 66 mm (2.6 in) for piers in or near the embankment. Horizontal movements, however, were significantly larger for piers in or near the embankment, with an average movement of 127 mm (5 in), compared with only 28 mm (1.1 in) for piers away from the embankment. These data indicate that when designing bridge piers in or near the toe of embankments, more consideration needs to be given to the increased horizontal stresses in these areas.

Influence of Foundation Movements on Bridge Structures

As shown in table 3, the most frequently occurring kinds of structural damage were superstructure distress, abutment damage, horizontal displacement, and bearing damage. Table 9 shows that bridges having only abutment movements had a high frequency of abutment damage and superstructure distress. Superstructure distress also occurred frequently for bridges with pier movement only. Table 10 shows that more kinds of structural damage occur for those bridges with simultaneous vertical and horizontal movements. Horizontal displacement, abutment damage, and superstructure distress all occurred frequently for bridges with both vertical and horizontal movements. In contrast, structures experiencing vertical movement only had the lowest frequency of structural damage, with 20 structures having no damage at all.

Table 6.—Abutment movements by foundation type

Foundation	Movement	Frequency: Number of abutments	Frequency: Percent moved	Range	Average	Standard deviation
				Millimetres	*Millimetres*	*Millimetres*
Spread footings	All types	120	100			
	Vertical	115	95.8	2.5- 965.2	162.6	188
	Horizontal	44	36.7	7.6- 355.6	76.2	68.6
	Vertical and horizontal	39	32.5	12.7- 965.2	254	264.2
				7.6- 355.6	76.2	71.1
Piles	All types	123	100			
	Vertical	77	62.6	7.6-1280.1	149.9	205.7
	Horizontal	79	64.2	12.7- 457.2	88.9	88.9
	Vertical and horizontal	35	28.5	12.7-1280.1	165.1	246.4
				10.2- 457.2	104.1	114.3

1 mm=3.9 × 10^{-2} in

Table 7.—Pier movements

Movement	Frequency: Number of piers	Frequency: Percent moved	Range	Average	Standard deviation
			Millimetres	*Millimetres*	*Millimetres*
All types	196[1]	100			
Vertical	137	73.5	2.5-1066.8	63.5	106.7
Horizontal	73	37.2	2.5- 508	101.6	134.6
Vertical and horizontal	19	9.7	12.7- 348	61	86.4
			15.2- 508	94	147.3

[1] Includes seven piers that raised vertically. These piers are not included in the total with vertical movement.

1 mm=3.9 × 10^{-2} in

This same trend was evident with magnitudes of movements—even moderate differential vertical movements produced a low incidence of structural damage. Of the 41 bridges with maximum differential vertical settlements of less than 102 mm (4 in), 20 experienced no damage. Most of the remaining 21 bridges experienced primarily abutment damage—minor cracking and opening or closing of construction joints—and minor superstructure distress. However, some bridges with larger differential vertical movements surprisingly did not experience abutment damage. For differential vertical movements larger than 102 mm (4 in), superstructure distress was predominant. Vertical displacement and poor riding quality were common for differential vertical movements of 203 mm (8 in) and greater. However, only 11 of the 180 bridges considered had poor riding quality. This matter will be discussed later in this article.

It was more difficult to correlate structural damage with magnitudes of substructure movements for those bridges where vertical and horizontal movements occurred simultaneously because of the possible interaction of the two movements. However, some observations can be made. Abutment damage was more frequent for bridges with both vertical and horizontal movements less than 102 mm (4 in) than for bridges with movement in only one direction. More than one-half of the 29 bridges experiencing movement in both directions had abutment damage when the magnitude of the vertical component was less than 102 mm (4 in). Similarly, more than one-half of the 39 bridges had abutment damage when the magnitude of the horizontal component of movement was less than 102 mm (4 in). In addition, horizontal displacement and bearing damage were more frequent for bridges having both vertical and horizontal movements (fig. 1). A detailed review of the actual causes

Table 8.—Pier movements by foundation type

Foundation	Movement	Number of piers	Percent moved	Range	Average	Standard deviation
				Millimetres	*Millimetres*	*Millimetres*
Spread footings	All types	92				
	Vertical	67	72.8	5.1–1066.8	68.6	137.2
	Horizontal	36	39.1	7.6– 508	104.1	142.2
	Vertical and horizontal	11	12	7.6– 614.7	170.2	182.9
				5.1– 228.6	109.2	154.9
Piles	All types	93[1]				
	Vertical	56	60.2	2.5– 355.6	63.5	73.7
	Horizontal	38	40.9	2.5– 457.2	101.6	127
	Vertical and horizontal	6	6.8	12.7– 348	111.8	129.5
				25.4– 86.4	58.4	27.9

[1] Includes seven piers that raised vertically. These are not included for vertical movement.

1 mm=3.9 × 10^{-2} in

Bridges that experienced horizontal movement only or simultaneous horizontal and differential vertical movement had a high frequency of structural damage, even for small horizontal movements. This suggests that horizontal movements are more critical than vertical movements in causing structural damage. For those structures with horizontal movements only, small movements (less than 51 mm [2 in]) caused superstructure distress in more than two-thirds of the bridges. The bearings were also damaged in more than one-third of these bridges. Abutment damage and horizontal displacement occurred more frequently for horizontal movements in excess of 102 mm (4 in).

of the kinds of distress in the bridges revealed that the horizontal component of the movement was usually responsible for the damage. Thus, as suggested earlier, horizontal movements appear to be much more critical than differential vertical settlement in causing most kinds of structural distress. This agrees with findings in references 16 and 17.

Table 11 shows that abutment damage occurred most frequently in simple span bridges but rarely in continuous bridges. Superstructure distress was the most frequent structural damage for continuous bridges and the second most frequent for simple span bridges.

Table 9.—Structural damage from moving substructure elements

Structural damage	Moving element: Abutment only Number of bridges	Abutment only Percent of bridges	Pier only Number of bridges	Pier only Percent of bridges	Abutment and pier Number of bridges	Abutment and pier Percent of bridges
Abutment damage	33	36.3	2	7.1	11	22
Pier damage	1	1.1	5	17.9	6	12
Vertical displacement	12	13.2	5	17.9	6	12
Horizontal displacement	21	23.1	6	21.4	13	26
Superstructure distress	37	40.7	9	32.1	19	38
Damage to rails, curbs, sidewalks, parapets	10	11	3	10.7	3	6
Bearing damage	22	24.2	4	14.3	9	18
Poor riding quality	8	8.8	2	7.1	1	2
Not given or corrected during construction	4	4.4	2	7.1	1	2
None	6	6.6	0	0	16	32
Total number of bridges	91		28		50	

Table 10.—Structural damage from movements

Structural damage	Movement: Vertical Number of bridges	Vertical Percent of bridges	Horizontal Number of bridges	Horizontal Percent of bridges	Vertical and horizontal Number of bridges	Vertical and horizontal Percent of bridges
Abutment damage	13	18.8	11	26.8	22	37.3
Pier damage	2	2.9	4	9.8	6	10.2
Vertical displacement	12	17.4	2	4.9	9	15.3
Horizontal displacement	3	4.3	12	29.3	25	42.4
Superstructure distress	21	30.4	22	53.7	22	37.3
Damage to rails, curbs, sidewalks, parapets	4	5.8	2	4.9	10	16.9
Bearing damage	1	1.4	16	39	18	30.5
Poor riding quality	7	10.1	0	0	4	6.8
Not given or corrected during construction	2	2.9	4	9.8	1	1.7
None	20	29	0	0	2	3.4
Total number of bridges	69		41		59	

Bearing damage was also frequent for continuous bridges but occurred in few of the simply supported bridges.

The analysis of structural distress, in terms of moving foundation elements, indicated differences between simple and continuous bridges. For the 33 simply supported bridges with abutment movement only, abutment damage and superstructure distress were the most common kinds of structural damage. However, for the 23 continuous bridges with abutment movement only, bearing damage was most frequent. Horizontal displacement and superstructure distress each occurred in over 30 percent of these bridges. However, for bridges with both abutment and pier movement, those 26 bridges with simple spans had a high occurrence of horizontal displacement and superstructure distress. For the 12 continuous bridges having both abutment and pier movement, the most prominent structural damage was superstructure distress.

Sample groups for structures with pier movement only were too small to validly compare. For some unknown reason, bridges with no structural damage were more frequent where both abutments and piers moved, regardless of span type. For both kinds of span, the most frequent and serious types of structural damage were related to horizontal movements.

Table 12 summarizes the frequency of bridge damage in terms of structural material. These data show that

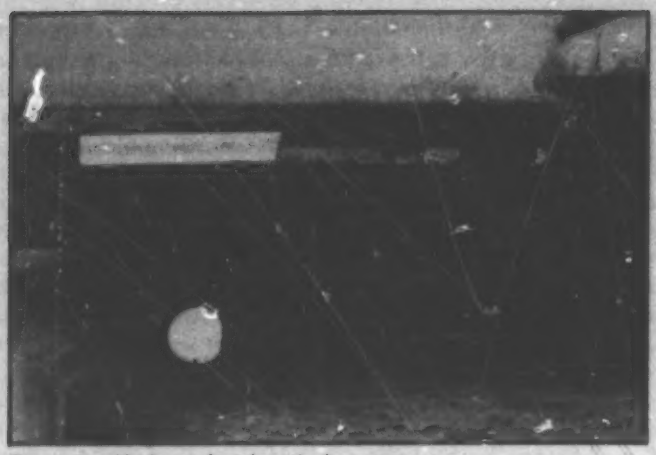


Figure 1.—Horizontal and vertical movements.

superstructure distress was reported much more often for concrete bridges than for steel bridges. Slightly more than one-half of the 55 concrete bridges experienced superstructure distress compared with less than one-quarter of the 73 steel bridges. However, the steel bridges had more abutment damage, horizontal displacement, and bearing damage. Overall, steel bridges, particularly those with differential vertical movement only (fig. 2), had less frequent and severe structural damage than did the concrete bridges. Of the 29 steel bridges that experienced only vertical movements, 41.4 percent had no noticeable damage, while more than one-half of the 23 concrete bridges with the same type of movement experienced superstructure distress. Over two-thirds of the steel bridges that had vertical movements of both abutments and piers experienced no structural damage. Again, it was found that even small horizontal movements—51 mm (2 in) or less—caused more frequent and severe structural damage than did much larger differential vertical movements, regardless of the structural material.

Table 13 shows the frequency of structural damage in terms of abutment type. These data show that bridges on full height abutments had the most abutment damage, but the least superstructure distress, bearing damage, and vertical and horizontal displacement. Although bridges on perched abutments had more serious structural damage, they experienced less structural damage in general. This is somewhat of a paradox; as reported earlier, perched abutments experienced a larger and wider range of movements than did full height abutments. However, a detailed examination of the data revealed that primarily differential vertical abutment movements of less than 102 mm (4 in) caused no damage to these bridges with perched abutments. The most damage was caused primarily by horizontal movements between 25 mm (1 in) and 102 mm (4 in), especially when these horizontal movements were accompanied by differential vertical movements greater than 102 mm (4 in). The high vertical movements experienced by the spill-through abutments (table 5) were largely responsible for the high incidence of superstructure distress.

Tolerance of Bridges to Foundation Movements

Movements were considered tolerable for slightly over one-half of the 154 bridges studied. Table 14 shows that of all the structural damage associated with tolerable foundation movements, abutment damage and superstructure distress occurred most frequently. In most instances, the reported damage involved minor cracking, the opening or closing of construction joints in the abutments, and cracking and spalling of concrete decks. The foundation movements of the 28 bridges that experienced no structural damage obviously were considered tolerable, accounting for 35 percent of the total structures with tolerable movements.

Almost one-half of the 74 bridges with intolerable movements had superstructure distress. Horizontal displacement and bearing damage also occurred frequently. In addition, over 25 percent of those bridges with intolerable movements had abutment damage. As expected, a larger number of bridges having intolerable movements had multiple structural damage than did the bridges having tolerable movements. The most frequent combinations of intolerable structural damage were superstructure distress, horizontal displacement, and bearing damage. The kinds of structural damage occurring most frequently were related to horizontal

Table 11.—Relationship of structural damage to span type

	Span type			
	Simple		Continuous	
Structural damage	Number of bridges	Percent of bridges	Number of bridges	Percent of bridges
Abutment damage	23	32.4	5	11.1
Pier damage	5	7	6	13.3
Vertical displacement	9	12.7	7	15.6
Horizontal displacement	15	21.1	9	20
Superstructure distress	20	28.2	17	37.8
Damage to rails, curbs, sidewalks, parapets	5	7	9	20
Bearing damage	10	14.1	13	28.9
Poor riding quality	2	2.8	6	13.3
Not given or corrected during construction	2	2.8	4	8.9
None	18	25.4	7	15.6
Total number of bridges	71		45	

Table 12.—Frequency of bridge damage by material type

Structural damage	Material			
	Steel		Concrete	
	Number of bridges	Percent of bridges	Number of bridges	Percent of bridges
Abutment damage	24	32.9	16	29.1
Pier damage	4	5.5	5	9.1
Vertical displacement	12	16.4	9	16.4
Horizontal displacement	20	27.4	12	21.8
Superstructure distress	17	23.3	28	50.9
Damage to rails, curbs, sidewalks, parapets	5	6.8	9	16.4
Bearing damage	19	26	10	18.2
Poor riding quality	4	5.5	3	5.5
Not given or corrected during construction	3	4.1	2	3.6
None	15	20.5	7	12.7
Total number of bridges	73		55	

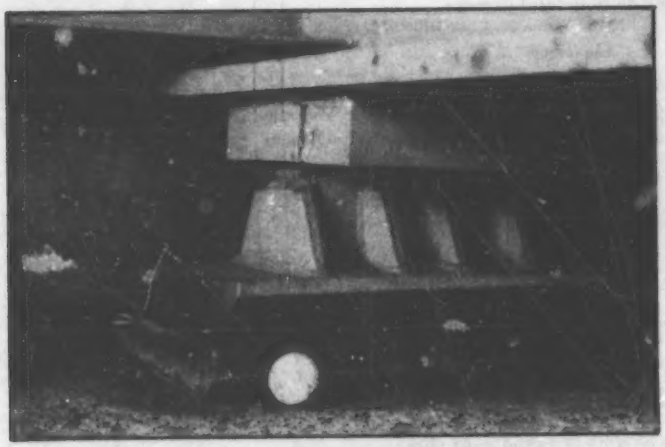


Figure 2.—Normal bearing device (above), and a steel girder bridge supported by a fabricated bearing rocker that has been shimmed 152 mm (6 in) (below).

movements or horizontal movements combined with vertical movements. The most frequently occurring sequence of events was the inward horizontal movement of abutments, which jammed the beams or girders against the back wall of the abutments, closed the expansion joints in the deck, and seriously damaged the bearings.

Because of poor riding quality on approaches to many bridges, riding quality was initially identified as one of the major areas of emphasis in the evaluation of tolerable bridge movements. However, poor riding quality was reported for only 11 actual bridge structures, and was intolerable for 10 of these. Of these 10 bridges, maximum differential vertical settlement ranged from 61 mm to 889 mm (2.4 in to 35 in), with an average of 353 mm (13.9 in). The maximum longitudinal angular distortion (maximum differential vertical settlement divided by the span length) ranged from 0.0077 to 0.063, with an average of 0.0203. Even the smallest of these values is larger than what reasonably might be expected to be tolerable from either a stress or serviceability standpoint. In other words, data indicate that the foundation movements would become intolerable for some other reason before creating intolerable rider discomfort. Consequently, riding quality probably will not be seriously considered when establishing tolerable movement criteria for highway bridges. This finding was based on static analysis and can change if dynamic aspects are found to govern this situation.

Intolerable movements generally were substantially larger than tolerable movements (table 15). Table 16 shows that moderate magnitudes of differential vertical movements were usually tolerable, while simultaneous vertical and horizontal movements were usually intolerable. All 28 of the differential vertical movements less than 51 mm (2 in) and 95 percent of those less than 102 mm (4 in) were tolerable. Although some larger differential vertical movements were tolerable, generally the tolerance to differential vertical movements decreased significantly for values over 102 mm (4 in). Only 50 percent of the differential vertical movements between 102 mm (4 in) and 203 mm (8 in) and 18 percent of those over 203 mm (8 in) were tolerable. Generally, maximum horizontal movements less than 51 mm (2 in) were tolerable and maximum horizontal movements of 51 mm (2 in) and greater were intolerable. Furthermore, table 16 shows that horizontal movements less than 51 mm (2 in) were tolerable in only 55 percent of the bridges when accompanied by differential vertical movements. Detailed data analysis showed that for simultaneous horizontal and vertical movements, horizontal movements were tolerable only when their magnitudes were 25 mm (1 in) and less.

Table 13.—Frequency of bridge damage by abutment type

Structural damage	Abutment					
	Full height		Perched		Spill-through	
	Number of bridges	Percent of bridges	Number of bridges	Percent of bridges	Number of bridges	Percent of bridges
Abutment damage	26	63.4	11	12.8	6	19.4
Pier damage	5	12.2	6	7	3	9.7
Vertical displacement	2	4.9	14	16.3	4	12.9
Horizontal displacement	4	9.8	21	24.4	10	32.3
Superstructure distress	12	29.3	30	34.9	17	54.8
Damage to rails, curbs, sidewalks, parapets	4	9.8	8	9.3	5	16.1
Bearing damage	5	12.2	22	25.6	5	16.1
Poor riding quality	1	2.4	7	8.1	1	3.2
Not given or corrected during construction	1	2.4	2	2.3	3	9.7
None	1	2.4	24	27.9	1	3.2
Total number of bridges	41		86		31	

Table 14.—Tolerance of bridges to structural damage

Structural damage	Movement					
	Tolerable			Intolerable		
	Number of bridges	Percent of bridges[1]	Multiple damages[2]	Number of bridges	Percent of bridges	Multiple damages
Abutment damage	22	27.5	9	19	25.7	16
Pier damage	3	3.8	3	8	10.8	6
Vertical displacement	7	8.8	5	15	20.3	13
Horizontal displacement	7	8.8	4	29	39.2	24
Superstructure distress	17	21.3	6	36	48.6	29
Damage to rails, curbs, sidewalks, parapets	5	6.3	4	9	12.2	7
Bearing damage	5	6.3	2	24	32.4	21
Poor riding quality	1	1.3	1	10	13.5	6
Not given or corrected during construction	3	3.8	1	4	5.4	0
None	28	35	0	0	0	0
Total number of bridges	80			74		

[1] Percent of bridges with indicated structural damage.

[2] Multiple damages refers to the number of bridges that had structural damage in addition to the indicated damage.

Although the sample sizes were smaller, the same trends shown in table 16 and described above held, regardless of span type (simply supported or continuous) and structural materials (steel or concrete). However, the lack of tolerance to horizontal movements was slightly more evident for all continuous structures and for concrete bridges. When the tolerance data were analyzed by number of spans the same trends held, but the sample sizes were often too small to be statistically reliable.

When the data shown in table 16 were analyzed by abutment type, some differences in tolerance to foundation movements became evident. The data clearly showed that bridges with full height abutments were more tolerant to both differential vertical and horizontal movements than bridges with either perched or spill-through abutments. For example, for full height abutments, 94.1 percent of the differential vertical movements less than 152 mm (6 in) were reported as tolerable. Although all of the differential vertical movements less than 102 mm (4 in) were reported as tolerable for bridges with perched abutments, only 18.2 percent of those greater than 102 mm (4 in) were considered tolerable. Similar trends were observed with horizontal movements. An unsuccessful attempt was

made to explain the significant findings among abutment types by correlating tolerance to movement with the various design and construction parameters used to select the most appropriate abutment for a particular bridge.

The influence of span length on the tolerance of bridges to foundation movements was studied in terms of maximum longitudinal angular distortion. Table 17 summarizes magnitudes of angular distortion considered tolerable and intolerable for all kinds of bridges included in this portion of the study and for subdivisions by span type and material type. When all of the bridges in the analysis are considered, 100 percent of the angular distortions less than 0.001 and 96.6 percent of the 58 angular distortions less than 0.005 are tolerable. However, only 36.4 percent of the values of angular distortion between 0.005 and 0.01 and 25 percent of those over 0.01 are tolerable. Therefore, an upper limit on angular distortion of 0.005 would be reasonable. For example, differential settlements of 76 mm (3 in) and 152 mm (6 in) would be tolerable most probably for spans of 15.2 m (50 ft) and 30.5 m (100 ft), respectively.

However, when the data are subdivided by span type, table 17 shows that the continuous bridges are more sensitive to angular distortion than are the simply supported bridges. Although this was expected, the difference was not as great as anticipated. For the continuous bridges 96.3 percent of the angular distortions less than 0.004 were tolerable, while only 28.6 percent of those over 0.004 were tolerable. In contrast, for the simply supported bridges, 100 percent of the angular distortions less than 0.005 were tolerable and no

Table 15.—Tolerance to movements

Tolerance to movements	Movement	Frequency: Number of bridges	Frequency: Percent moved	Range	Average	Standard deviation
				Millimetres	*Millimetres*	*Millimetres*
Tolerable	All types	72	100			
	Vertical	44	61.1	0–457.2	61	88.9
	Horizontal	12	16.7	7.6–228.6	66	73.7
	Vertical and horizontal	16	22.2	0–304.8 7.6–152.4	61 43.2	86.4 43.2
Intolerable	All types	71	100			
	Vertical	16	23.9	58.4–1066.8	320	312.4
	Horizontal	20	26.8	25.4–457.2	182.9	129.5
	Vertical and horizontal	35	49.3	0–1280.2 10.2–508	276.9 127	335.3 119.4

1 mm=3.9 × 10^{-2} in

Table 16.—Range of movement magnitudes considered tolerable or intolerable

Movement interval[1]	Number of bridges with given type of movement: Vertical only		Horizontal only		Vertical and horizontal: Vertical component		Vertical and horizontal: Horizontal component	
	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable
Millimetres								
0–50.8	28	0	8	1	10	5	11	9
50.8–101.6	9	2	1	7	4	9	3	10
101.6–152.4	2	3	1	0	0	4	1	6
152.4–203.2	3	2	0	3	0	3	1	3
203.2–254	0	3	2	4	1	1	0	3
254–381	1	2	0	3	1	6	0	2
381–508	1	2	0	2	0	2	0	1
508–1524	0	2	0	0	0	5	0	1
Total	44	16	12	20	16	35	16	35

[1] For vertical movements, magnitudes refer to maximum differential vertical movement. For horizontal movements, magnitudes refer to maximum horizontal movement of a single foundation element.

1 mm=3.9 ×10^{-2} in

Table 17.—Tolerable or intolerable magnitudes of longitudinal angular distortion

Angular distortion interval ($\times 10^{-3}$)	Number of bridges of given type and tolerance									
	All bridges		Span				Material			
			Simple		Continuous		Concrete		Steel	
	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable
0–0.99	31	0	13	0	18	0	20	0	11	0
1–1.99	8	1	5	0	3	1	5	1	3	0
2–2.99	6	0	3	0	3	0	3	0	3	0
3–3.99	7	0	5	0	2	0	2	0	5	0
4–4.99	4	1	3	0	1	1	3	1	1	0
5–5.99	0	1	0	0	0	1	0	0	0	1
6–6.99	1	2	0	1	1	1	1	0	0	2
7–7.99	1	2	1	1	0	1	1	0	0	0
8–8.99	0	0	0	0	0	0	0	0	0	0
9–9.99	2	2	1	1	1	1	1	2	1	0
10–19.9	3	6	2	3	1	3	2	2	1	4
20–39.9	1	5	1	4	0	1	1	2	0	3
40–59.9	0	0	0	0	0	0	0	0	0	0
60–79.9	0	1	0	0	0	1	0	0	0	0
Total	64	21	34	10	30	11	39	8	25	10

angular distortion was intolerable until the value was greater than 0.006. For the above example, differential settlements of 61 mm (2.4 in) and 122 mm (4.8 in) would be more reasonable tolerable limits for continuous spans of 15.2 m (50 ft) and 30.5 m (100 ft), respectively.

Table 17 suggests that a maximum angular distortion of 0.005 would be a reasonable limit for both concrete and steel bridges. The reported trend (table 12) for the concrete bridges to experience more frequent and severe superstructure damage than the steel bridges as a result of foundation movements was not supported by the tolerance data in table 17. This implies that the frequently reported superstructure distress of concrete bridges was quite often determined to be tolerable. A detailed analysis of the data in table 14 by material type verified this.

Summary

The data presented above show that a range of both vertical and horizontal movements of substructure elements has been experienced by a substantial number of U.S. and Canadian highway bridges. Generally, abutment movements occurred more frequently than pier movements. Although both the frequency and magnitude of vertical movements were substantially greater than horizontal movements for both abutments and piers, the horizontal movements were more damaging to bridge superstructures.

There were more pile foundations than spread footing foundations for both abutments and piers. Although spread footing foundations moved more frequently and with greater magnitudes than pile foundations, the differences were not large. It can be concluded that pile foundations do not always eliminate foundation movement and therefore are not necessarily superior to spread footings.

Depending upon the span type, span length, and construction material, the field data show that many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without sustaining serious structural damage. Horizontal movement of substructure elements, particularly abutments, was found to be much more damaging than vertical movement and must be closely controlled to avoid serious structural damage. These results of the field data analysis will be combined with the results of the analytical studies of the behavior of bridges subjected to movements and used to develop a methodology for designing bridges to provide a rational set of criteria for tolerable bridge movements.

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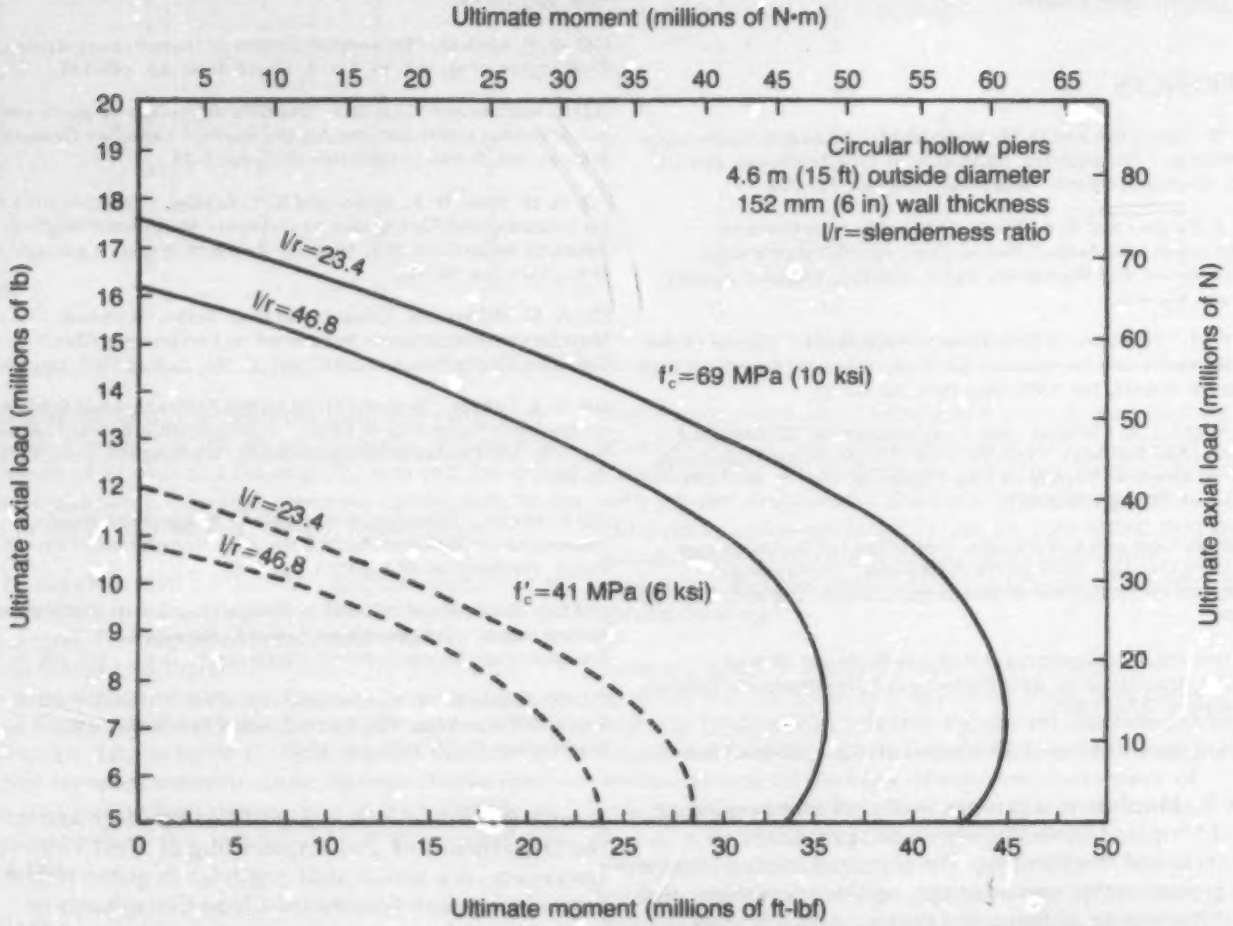
Lyle K. Moulton is a professor of civil engineering at West Virginia University where he specializes in geotechnical engineering. His principal areas of interest are ground water and seepage, earthwork design, and foundation engineering. His recent research work includes studies on in situ permeability measurement and tolerable bridge movements.

Joseph R. Kula, formerly a graduate research assistant in the Department of Civil Engineering at West Virginia University, is a senior staff engineer in geotechnical engineering with Woodward-Clyde Consultants in Rockville, Md.

Applications of High Strength Concrete for Highway Bridges

by
J. E. Carpenter


Ultimate moment (millions of N·m)
5 10 15 20 25 30 35 40 45 50 55 60 65
20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5
Circular hollow piers
4.6 m (15 ft) outside diameter
152 mm (6 in) wall thickness
l/r=slenderness ratio
l/r=23.4
l/r=46.8
f'c=69 MPa (10 ksi)
l/r=23.4
l/r=46.8
f'c=41 MPa (6 ksi)
80 70 60 50 40 30 20 10
Ultimate axial load (millions of lb)
Ultimate axial load (millions of N)
0 5 10 15 20 25 30 35 40 45 50
Ultimate moment (millions of ft-lbf)

Over the years the practical design compressive strength of portland cement concrete has steadily increased from 21 MPa (3 ksi) to 41 MPa (6 ksi). Now it is feasible to produce and design for 69 MPa (10 ksi) concrete. Concrete of this strength requires careful selection of high quality mixing materials and more rigid quality control. High strength concrete has been used successfully in the columns of a 76-story office building and in a 140 m (461 ft) liquid petroleum gas storage vessel.

High strength concrete has been made and used by prestressed concrete producers for years. However, the producers use the high *early* strength of the concrete so that the formwork may be removed, the concrete members prestressed, and the forms reused every day—without taking advantage of the eventual 55 to 69 MPa (8 to 10 ksi) compressive strength in the structural design.

This article describes the results of the first task of a three-task Federal Highway Administration (FHWA) research project to identify optimum structural applications of high strength concrete (approximately 69 MPa [10 ksi] compressive strength) in highway bridges. Task A,

Analytical Design Study, has been completed. Mixing specifications, physical properties, and advantages and disadvantages of high strength concrete were studied. Based on this information and analytical design studies, applications were developed for using high strength concrete in various kinds of bridges. Task A recommended a test program that will be performed and interpreted in Task B, Structural Laboratory Testing. Task C, Revisions to Design Criteria, will revise current bridge design criteria as necessary to implement the use of high strength concrete.

Mixing and Placing High Strength Concrete

Specifications for mixing and placing high strength concrete are based on the following: *(1-3)*[1]

Constituents: Cement, aggregates, and admixtures that will produce concrete of the required strength and insure consistency of properties over a period of time must be selected. The maximum size of coarse aggregate must be small, usually not greater than 13 mm (0.5 in). Superplasticizers may be used.

Water/cement ratio: The water/cement ratio must be low, usually between 0.30 and 0.35, by weight.

Cement content: The cement content must be high, usually between 475 and 565 kg/m³ (800 and 950 lb/yd³).

Compaction: Good compaction, combined with sturdy forms, is necessary to produce adequately dense concrete.

Quality control: A well-managed quality control program, including planning and cooperation of all parties involved, is essential to success.

[1] Italic numbers in parentheses identify references on page 83.

Physical Properties of High Strength Concrete

The properties of high strength concrete, similar to the properties of normal strength concrete, can be predicted by anyone familiar with concrete engineering. Ratios of tensile strength to square root compressive strength for high strength concrete are similar to those obtained for normal strength concrete. *(4)* Stress-strain relationships for high strength concrete are shown in figure 1. *(4-6)* The initial slope of the curves tends to increase with increasing compressive strength (modulus of elasticity increases), and the strain at maximum stress tends to increase slightly. Also, the slope of the descending portion of the curves is steeper for the high strength concretes than for normal strength concretes, so the high strength concrete in effect is more brittle.

Data indicate that modulus of elasticity for 69 MPa (10 ksi) concrete is represented better by the formula $E_c = 26w^{1.5}\sqrt{f'_c}$ $(0.0337w^{1.5}\sqrt{f'_c})$ *(4, 6, 7)* than by the formula $E_c = 33w^{1.5}\sqrt{f'_c}$ $(0.0428w^{1.5}\sqrt{f'_c})$ presently suggested by the American Association of State Highway and Transportation Officials (AASHTO) code and the American Concrete Institute (ACI) code and used for normal strength concrete. *(8, 9)* High strength concretes creep less than normal strength concretes when loaded to a given percentage of compressive strength. *(4)* Shrinkage of high strength concrete is about the same as that of normal strength concrete. *(4, 7)*

Advantages and Disadvantages of High Strength Concrete

One advantage of high strength concrete is its greater compressive strength, which can be evaluated in relation to unit cost, unit weight, and unit volume. High strength concrete, with its greater compressive strength per unit cost, is the least expensive means of carrying compressive force. In addition, its greater compressive strength per unit weight and unit volume allows lighter, more slender bridge piers, providing improved horizontal clearances.

Other advantages of high strength concrete include increased modulus of elasticity and increased tensile strength. Increased stiffness is advantageous when deflections or stability govern the bridge design, and increased tensile strength is advantageous in service load design in prestressed concrete.

A disadvantage of high strength concrete is that the mix has much less water than normal strength concrete. This results in mixes that have reduced workability and handling time, making them more difficult to place and properly compact. In addition, obtaining high quality aggregates in necessary sizes and obtaining cement that will consistently produce concrete of the required strength may be difficult.

Structural design considerations may preclude effective use of the increased concrete strength. Cross section dimensions often are governed by factors other than stress, such as minimum cover, so that the full strength capability of the concrete is not used. Further, the total prestress force that can be generated may not be sufficient to take advantage of the high strength concrete.

Other disadvantages of high strength concrete are its additional cost and the additional expenses of increased quality control. Finally, AASHTO specifications tend to discourage the use of high strength concrete because the specifications are based on the properties of normal strength concrete. *(8)*

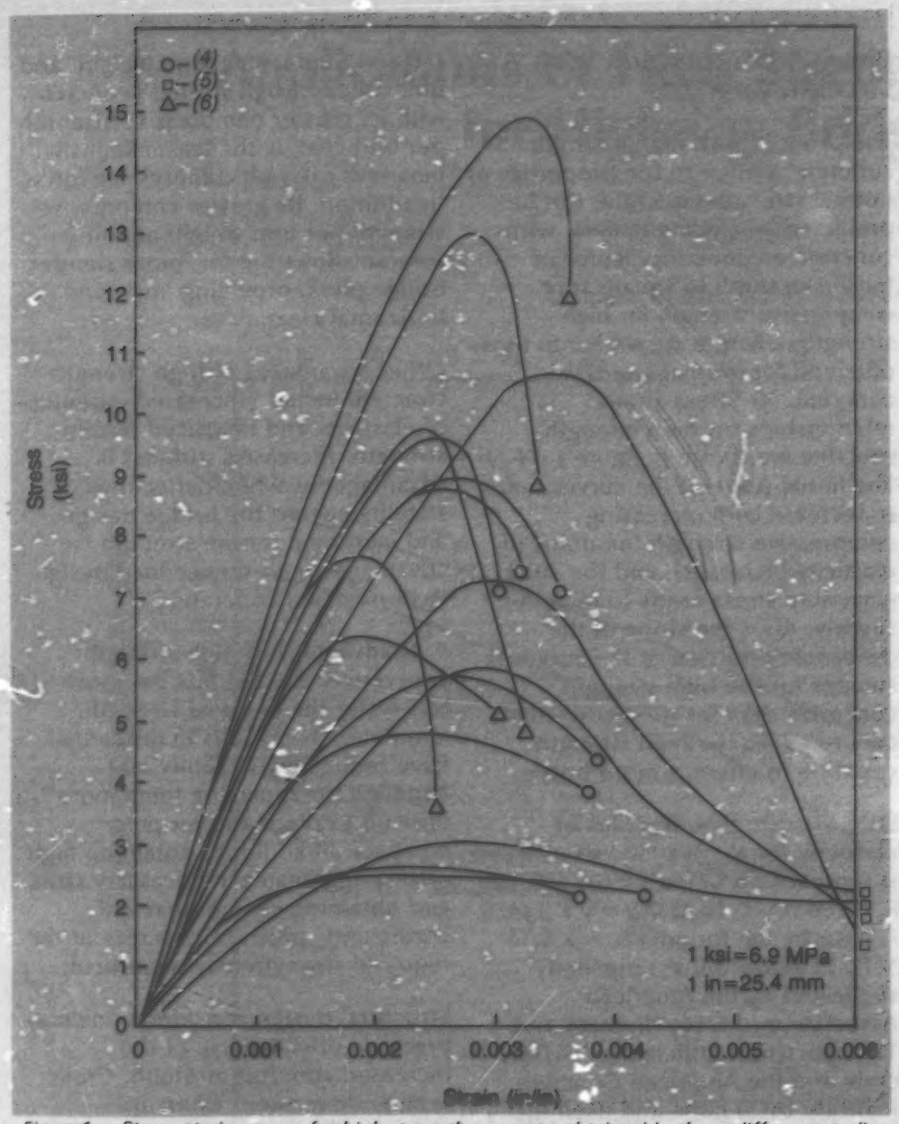


Figure 1.—Stress-strain curves for high strength concrete obtained in three different studies.

Applications of High Strength Concrete

Solid section girders

The effect of using high strength concrete in four different solid section girders was investigated. Cross section dimensions of the four girders—the bulb tee, the Washington State Department of Transportation (WSDOT) Standard, the AASHTO-Prestressed Concrete Institute (PCI) Standard, and the Colorado Standard—are shown in figure 2. Two methods of attaching the decks were considered: integrally cast and cast in place. Decks were cast in place on the completed girder without shoring, so that the entire dead load of both girder and deck was carried by the girder section.

To determine span capabilities of the girders, the AASHTO HS20–44 loading was used, with a lateral distribution factor S/5.5. Allowable stresses conformed to the AASHTO code, with allowable tension in the precompressed tension zone equal to $3\sqrt{f'_c}$ ($0.249\sqrt{f'_c}$) and allowable compression equal to $0.4\ f'_c$.

The span capabilities for a 1829 mm (72 in) integral deck bulb tee is shown in figure 3. Span capabilities at various girder spacings are given for concrete strengths of 41 MPa (6 ksi), 55 MPa (8 ksi), and 69 MPa (10 ksi). As shown, span capability for the closer spaced girders increases with each increase in concrete strength. For the wider spaced girders, capability increases when concrete strength is increased from 41 MPa (6 ksi) to 55 MPa (8 ksi). However, for concrete strengths above 55 MPa (8 ksi), span capability does not increase because insufficient prestress force inhibits the capabilities of the stronger concrete. Similar patterns were found for the other cross sections investigated.

Figure 3 also shows span capabilities based on a live load deflection criterion of l/800 (section 1.7.6 of AASHTO code for steel structures). *(8)* If applicable, this criterion governs most beam spacings.

Figure 4 shows span capabilities for the same 1829 mm (72 in) section with a cast-in-place deck. Although span capabilities increase with each increase in concrete strength, span capabilities are constantly lower for the cast-in-place deck than for the integral deck.

Making maximum use of the greater load carrying capabilities of high strength concrete girders requires different designs for bridges. Two designs of a 46 m (150 ft) simple span bridge are shown in figure 5. In the nine-girder design, 41 MPa (6 ksi) concrete girders were used. In the four-girder design, 69 MPa (10 ksi) concrete girders were used. The

advantages of the high strength concrete are evident: Only four girders are needed for the high strength concrete design, while nine girders are needed for the normal strength concrete design. In spite of the thicker cast-in-place deck needed for the greater transverse span between the four girders, the overall dead load is reduced, and therefore total prestressing requirements are reduced.

Posttensioned box girders

Multiple span, cast-in-place, continuous posttensioned box girder bridges of constant depth were represented by the two-span, continuous structure shown in figure 6. Concrete strengths were 41 MPa (6 ksi), 55 MPa (8 ksi), and 69 MPa (10 ksi). Overall beam depths were 1.4 m (4.5 ft), 1.7 m (5.5 ft), and 2 m (6.5 ft). Allowable stresses were the same as those used in the solid section girder analysis, except that allowable tension in the precompressed tension zone was assumed to be $6\sqrt{f'_c}$ ($0.498\sqrt{f'_c}$). Loading was three lanes of AASHTO HS20–44, without lane reduction.

Span capabilities for the different beam depths are shown in figure 7. High strength concrete for continuous box girders of 46 m to 76 m (150 ft to 250 ft) spans increased span capabilities. As with the integral deck solid section girders, the maximum available prestress force limited capabilities of the high strength concrete.

Segmentally posttensioned box girders

Segmentally posttensioned box girder bridges of medium to long span were represented by the recently completed free cantilever Shubenacadie Bridge (South Mainland, Nova Scotia) with a 213 m (700 ft) main span and 113 m (372 ft) side spans. *(10)* Overall dimensions

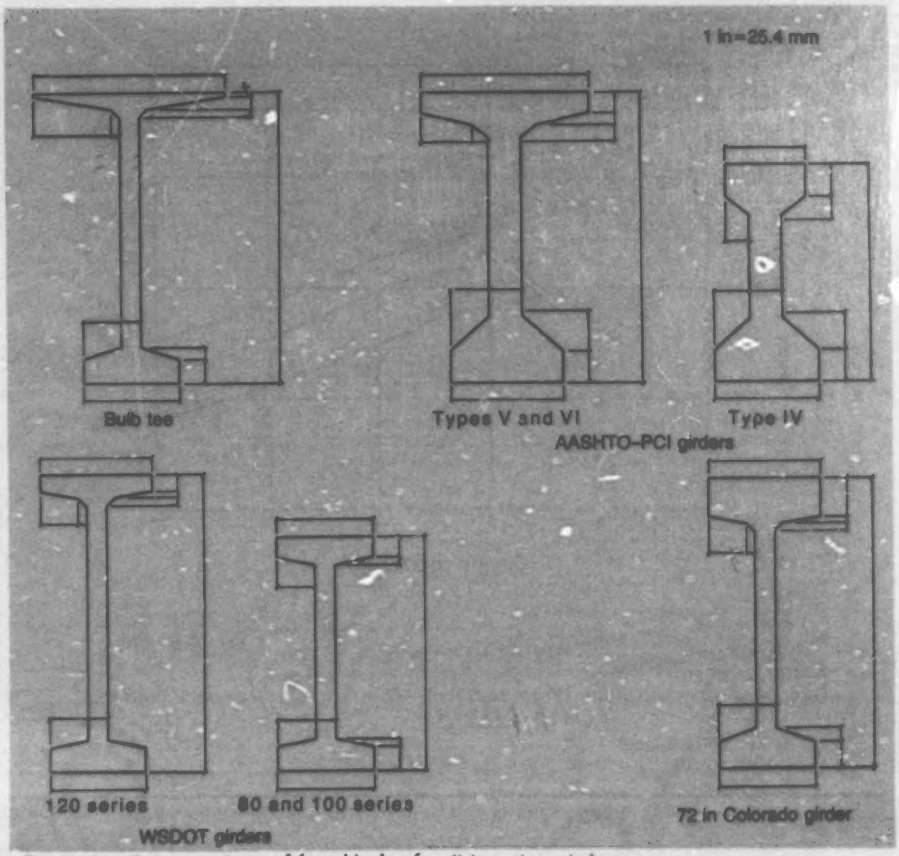
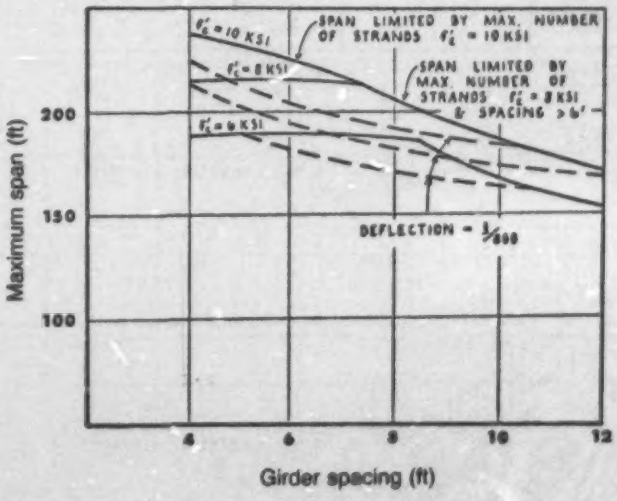


Figure 2.—Cross sections of four kinds of solid section girders.



1 ksi=6.9 MPa
1 ft=0.305 m

Figure 3.—Span capabilities for a 1829 mm (72 in) integral deck bulb tee.

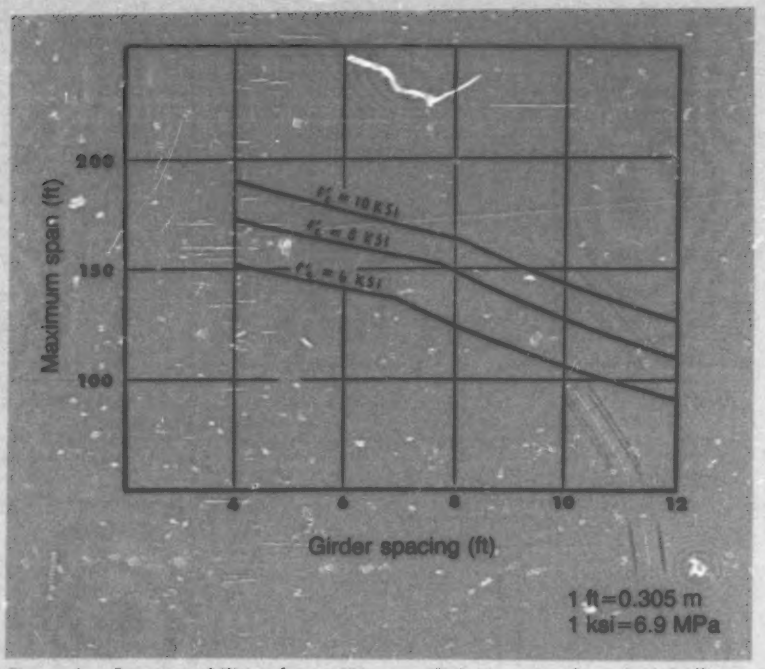


Figure 4.—Span capabilities for a 1829 mm (72 in) cast-in-place deck bulb tee.

of the bridge are shown in figure 8. The bridge was constructed with 34 MPa (5 ksi) concrete and used 31.75 mm (1.25 in) diameter thread bars for posttensioning. No tension was allowed in the precompressed tension zone. The AASHTO HS20–44 loading was used.

The bridge was redesigned with high strength concrete to determine how much the thickness of the lower flange could be reduced and what effect this reduction would have on the overall moments. As shown in figure 9, using high strength concrete reduced the total flexural prestress force by more than 10 percent as a result of the reduced dead load. The optimum lower flange thickness is about 0.5 m (1.6 ft), obtained at 55 MPa (8 ksi) strength.

Despite the large size of the Shubenacadie Bridge, geometric requirements for cover over reinforcement and prestressing still

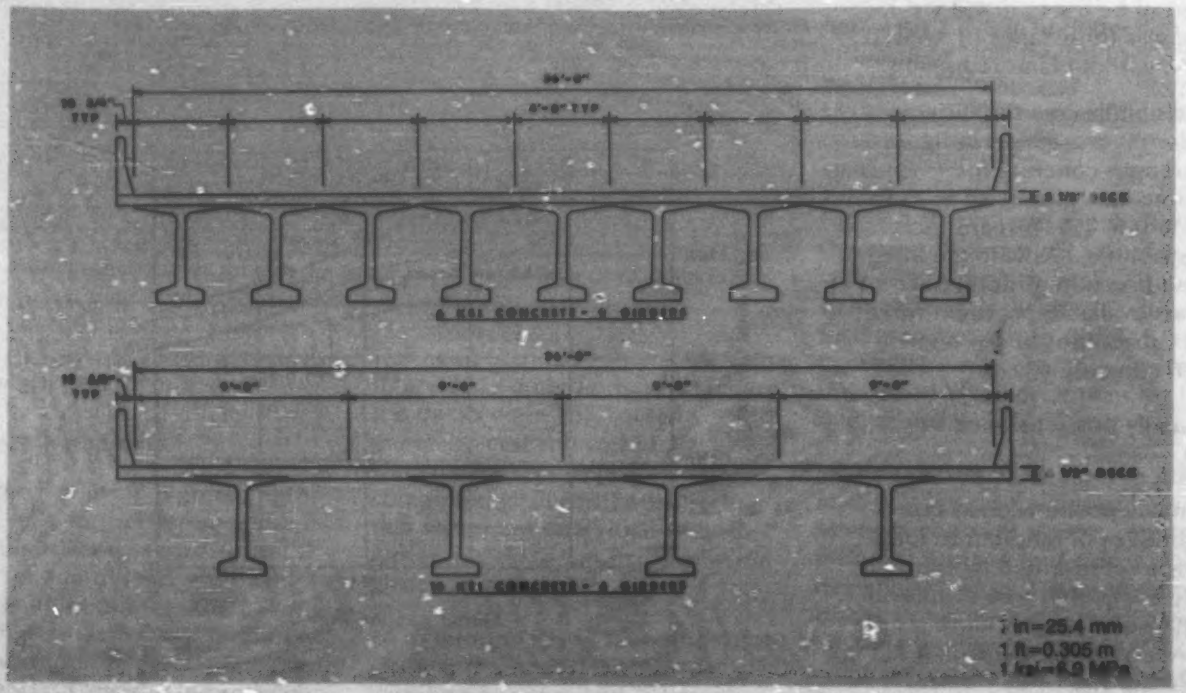


Figure 5.—Two 46 m (150 ft) simple span bridges with different concrete strengths.

govern thicknesses of deck and webs. By using high strength concrete in wider bridges with three or more webs, the number of webs can be decreased and the transverse span lengths of the deck increased, similar to the wider spacings used for the solid section girders made of high strength concrete. Transverse posttensioning is included in the design. The reduced weight and loads possible with high strength concrete further reduce concrete area and prestress force.

For segmentally posttensioned box girder bridges, high strength concrete is feasible in regions, such as the lower flange, where the design is controlled by stress. In regions, such as in the deck, where the design is controlled by other factors, normal strength concrete can be used. The webs may be constructed of either high strength concrete or normal strength concrete, depending on minimum thickness requirements, minimum shear reinforcement requirements, and the contribution of concrete in the webs to the shear carrying capacity.

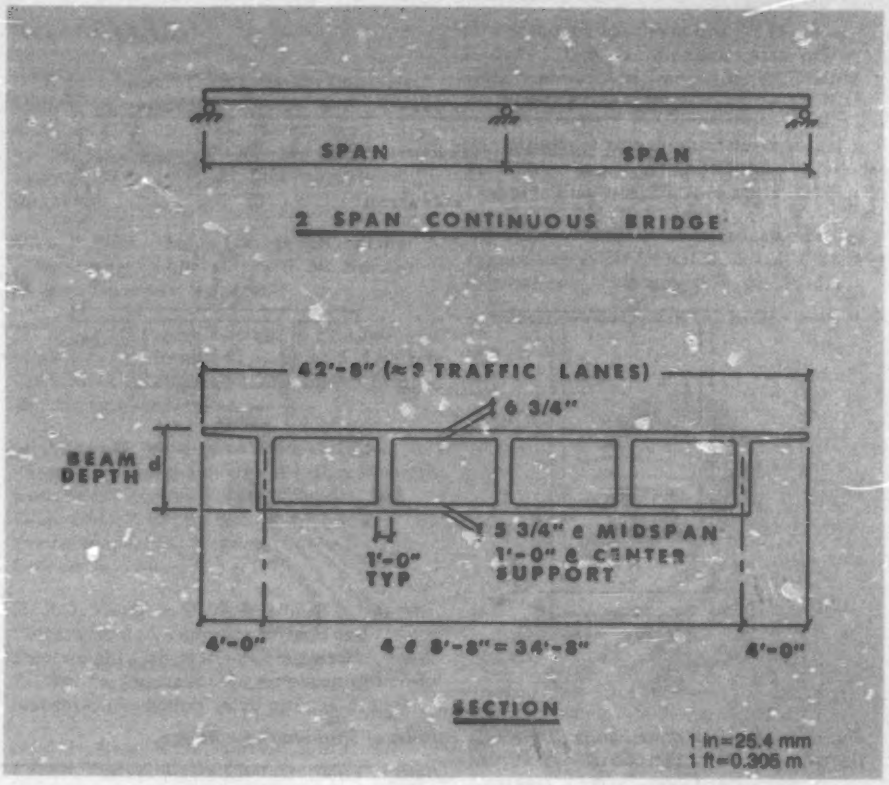


Figure 6.—Two-span continuous posttensioned box girder bridge.

Compression members

Pier shafts and elements of Y-piers (fig. 10) are examples of compression members in bridges. Compression members were expected to benefit greatly from high strength concrete.

To study the effect of increasing the compressive strength of concrete from 41 MPa (6 ksi) to 69 MPa (10 ksi), interaction diagrams were developed for the compression member shown in figure 11. The diagrams were developed for the pin end condition, assuming that the cross section possessed the bilinear moment-curvature relationship defined by strain compatibility and the elastic properties of the concrete and prestressing strand. The deflected shapes were determined by integrating the curvatures along the

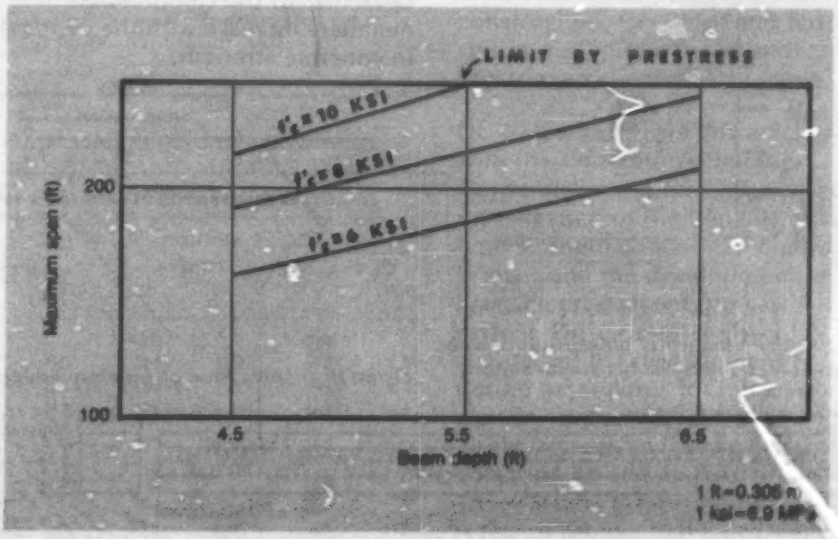


Figure 7.—Span capabilities for a two-span continuous box girder bridge with different beam depths.

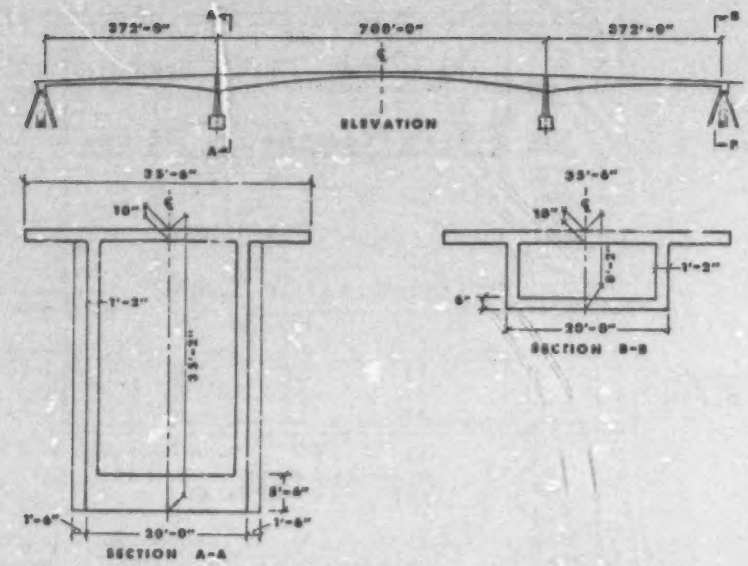


Figure 8.—Overall dimensions of the free cantilever Shubenacadie Bridge.

length of the member. The ultimate strength design method was used with factored loads.

Figures 12 and 13 show that capacities of the compression members increase with the increase in concrete strength.

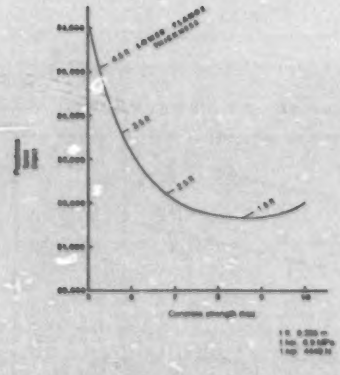


Figure 9.—Variation of prestress force with concrete strength (Shubenacadie Bridge).

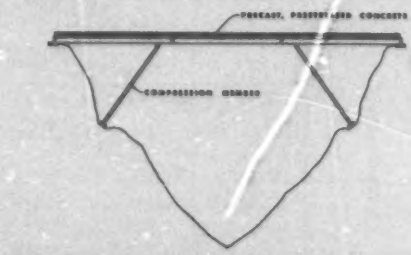


Figure 10.—Application of Y-piers in bridges.

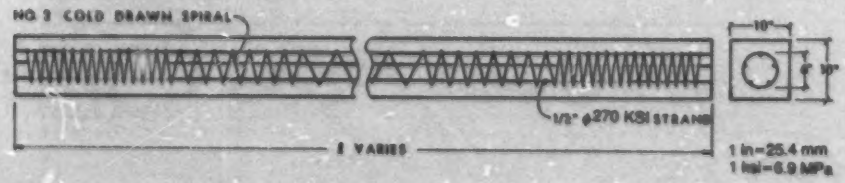


Figure 11.—Compression member details.

Results of the compression member study are shown in table 1. Two slenderness ratios, three concrete strengths, and three end eccentricities are shown. For short, concentrically loaded struts, capacity is determined by multiplying the cylinder strength by the cross sectional area and then subtracting the prestress force. Because the prestress force is constant, the benefits of increasing concrete strength are more than the strength increase itself. This is illustrated by the strength ratios in the table that exceed 10/6=1.67 for those struts.

For combined axial load and bending, using high strength concrete, even for relatively slender columns, is beneficial. It can be concluded that compression members are an excellent application for high strength concrete. Smaller sections can be used for a given number of members, or fewer members can be used in a given location. In either case, weight as well as material and construction cost are reduced.

Summary

The analytical design studies discussed above demonstrate the benefits of using high strength concrete in solid section girders, posttensioned box girders, segmentally posttensioned box girders, and compression members. These benefits include increased span lengths, reduced dead loads, and greater load capacities.

Continuing research under this project will provide additional design information on potential structural applications for high strength concrete as well as evaluation of data obtained from laboratory tests of structural specimens.

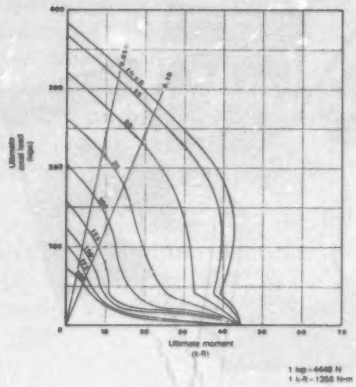


Figure 12.—Interaction diagram for compression member of 41 MPa (6 ksi) concrete.

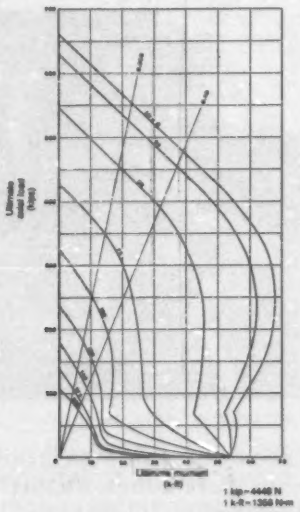


Figure 13.—Interaction diagram for compression member of 69 MPa (10 ksi) concrete.

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(9) "Building Code Requirements for Reinforced Concrete, ACI 318–77," *American Concrete Institute*, Detroit, Mich., 1977.

(10) D. H. Franklin, "Shubenacadie Bridge Tames Angry River," vol. 1, No. 2, *Concrete International*, February 1979, pp. 73–82.

Table 1.—Axial load capacities for a 254 mm (10 in) square strut

Slenderness ratio l/r	Concrete strength f'_c	Eccentricity e=0 P	e=0 P/P_o	e=0.1D P	e=0.1D P/P_o	e=0.2D P	e=0.2D P/P_o
	ksi	*kips*		*kips*		*kips*	
0	6	375	1.00	284	1.00	217	1.00
	8	518	1.38	389	1.37	294	1.35
	10	662	1.77	478	1.69	362	1.67
50	6	320	1.00	233	1.00	168	1.00
	8	437	1.37	317	1.36	220	1.31
	10	545	1.70	390	1.67	267	1.59
100	6	205	1.00	133	1.00	80	1.00
	8	270	1.32	165	1.24	90	1.13
	10	325	1.59	190	1.43	100	1.25

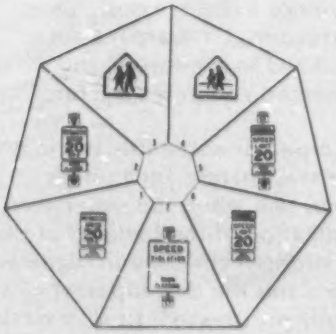
1 ksi = 6.9 MPa
1 kip = 4448 N

James E. Carpenter recently joined Andersen–Bjornstad–Kane–Jacobs, Inc., where he is project leader in charge of substructure design for the West Seattle Bridge in Washington State. In his 25 years experience in bridge design and construction, Dr. Carpenter has specialized in reinforced and prestressed concrete structures.

Dr. Carpenter wrote this article while with the Concrete Technology Corporation where he supervised an investigation of the benefits of very high strength concrete in highway bridges and the development of a biaxially prestressed, precast deck system. Dr. Carpenter also supervised the development of design procedures for integral bent caps in reinforced concrete box girder bridges and participated in construction and loading of the Three Sisters Bridge model—a research project that aided the acceptance of prestressed concrete free cantilever bridge construction in the United States.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Division. The reports are available from the address noted at the end of each description.



Effectiveness of Speed Control Signs in Rural School Zones and Small Communities, Report No. FHWA-RD-79-20

by FHWA Traffic Systems Division

This report assesses the effectiveness of various speed control signs located on high-speed, two-lane highways in rural school zones and small communities. Vehicle speed data were collected in two small communities in Mississippi and at six school zones in California, Oregon, and Mississippi. Existing signs in the test areas were evaluated in relation to the addition of a reduced speed ahead sign, beacon flashers coupled with the speed limit sign and reduced speed ahead sign, and a traffic-actuated speed violation sign. Roadside interviews were conducted to study driver understanding of and attitude toward the speed control devices. Vehicle speed data and interview data determined if each sign tested increased safety and driver awareness of potential hazards.

Results indicated there was a 3.22 km/h to 16.10 km/h (2 mph to 10 mph) reduction in average operating speed using beacon flasher signs and speed violation signs.

A limited number of copies of the report are available from the Traffic Systems Division, HRS-33, Federal Highway Administration, Washington, D.C. 20590.

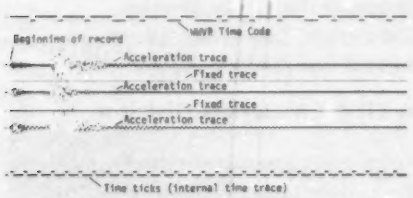
Identification of Traffic Management Problems in Work Zones, Report No. FHWA-RD-79-4

by FHWA Traffic Systems Division

This report presents a prioritized listing of 20 problem statements on traffic safety in construction, maintenance, and utility work zones. These problem statements were developed from data collected from more than 100 site visits. All aspects of work zone traffic control were examined including planning, design, installation, operation, maintenance, and removal. Detailed analyses were made of accidents in 30 construction zones to determine why accidents happen in work zones. From the site observations, accident analyses, and a literature review, the

principal areas of work zone traffic control needing research were defined. Results indicate that approximately two-thirds of the work zone safety problems would be less severe if current standards and knowledge were properly applied.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 80 139520).



Use and Interpretation of Strong-Motion Records From Highway Bridges, Report No. FHWA-RD-78-158

by FHWA Structures and Applied Mechanics Division

In recent years several State highway bridges in high seismic probability regions of the United States have been permanently instrumented with strong motion accelerometers and recorders to obtain a time-based record of any bridge dynamic structural motions caused by a local earthquake.

This report presents background information on strong motion instrumentation and strong motion records and describes analytical methods proposed for interpreting the complex data contained in records of the dynamic responses of moderate length bridges to strong ground shaking. A linear mathematical model is formulated for calculating the bridge responses. Techniques are outlined for reducing both analog and digital records of strong motion responses. The objective of the analysis of either type of record is to determine force levels throughout a given bridge for a particular earthquake input and to identify the mathematical model parameters that best describe the measured responses. A computerized linear least squares fitting procedure in the time domain is proposed to identify mathematical model parameters from recorded strong motions. In two example problems, force levels in a typical bridge are calculated for hypothetical strong motion inputs.

The appendixes to the report provide an extensive discussion of the dynamics of structures and a listing for the computer program used.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 292254).

Aerodynamic Investigations of the Luling, Louisiana Cable-Stayed Bridge, Report No. FHWA-RD-77-161

by FHWA Structures and Applied Mechanics Division

The selection of the final design for a proposed cable-stayed girder Interstate highway bridge over the Mississippi River at Luling, La., was preceded by a comprehensive wind tunnel section model study of the aerodynamic stability of seven alternate suspended superstructure configurations. This experimental study was conducted in the George S. Vincent Memorial Wind Tunnel at the FHWA Fairbank Highway Research Station in McLean, Va. The motion responses of the section models were evaluated over a range of wind velocities and aspect ratios to identify possible susceptibility of the prototype suspended span designs to excessive vertical or torsional vibrations from wind. After the relative stability of each of the seven configurations had been established, additional wind tunnel studies were conducted on models of three partially completed stages of construction of the preferred design and of a cable support tower to determine whether aerodynamic instability might occur at intermediate stages of construction. Wind flow patterns around the preferred suspended span design model were investigated to determine optimum locations for wind-monitoring sensors on the prototype structure.

The results of the study indicated that critical aerodynamic flutter should not occur with the preferred configuration of the suspended span at wind velocities up to 240 km/h (150 mph) and that significant vibrations from wind vortex shedding action should not occur if the damping ratio in the prototype is at least 1 percent of critical. None of the sequential construction stages of the prototype bridge suspended span or tower should show any aerodynamic instability in the same wind velocity

range. Drag forces measured on the models correlated with the American Association of State Highway and Transportation Officials design values. The results of the wind flow pattern study showed that if the wind sensors are mounted on top of 10.7 m (35 ft) light poles in the bridge roadway median, the measured ambient wind velocities and directions will be free of distortions caused by the boundary layer effect.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 292859).

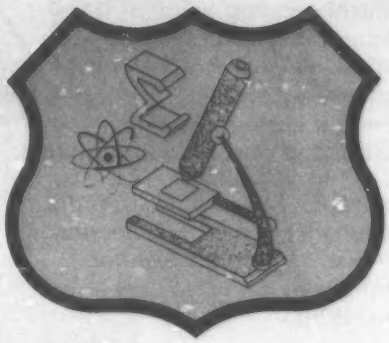
Coating and Corrosion Costs of Highway Structural Steel, Report No. FHWA-RD-79-121

by FHWA Materials Division

A simulation model has been developed to aid in selecting cost-effective maintenance policies for highway bridge painting. In addition to analyzing costs for a specific maintenance measure selected, the model can determine the optimal protection method and painting schedule for a specific bridge. This model shows that the most commonly used protection method—a 0.13 mm (5 mils) oleoresinous system—is not optimal. In a marine environment optimal protection is offered by a 0.18 mm (7 mils) zinc-rich system. In an industrial environment optimal results are obtained by either a 0.18 mm (7 mils) zinc-rich system or a 0.15 mm (6 mils) synthetic polymer system. The latter system is also optimal in a rural environment. According to data gathered in 1979, the minimum cost of adequately painting highway bridges under State jurisdiction is about $160 million. The actual amount spent is only about $80 million. The result is annual rehabilitation costs of $300 to $400 million.

The cost to the taxpayer for corrosion of the Nation's bridges has been estimated at about $120 to $150 million for 1979. This cost includes painting, inspection, contract preparation, and overhead but does not include rehabilitation costs.

The report is available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.



Alternate Highway Deicing Chemicals, Executive Summary, Report No. FHWA-RD-79-109

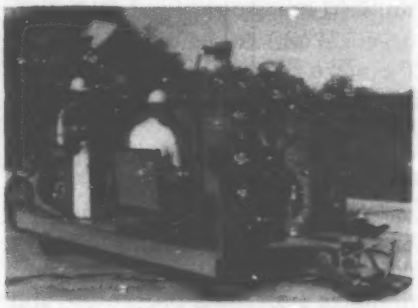
by FHWA Materials Division

Numerous chemical compounds with the potential to replace sodium chloride (NaCl) as a roadway deicer were studied because of drawbacks associated with using NaCl. Potential replacements were selected after considering their water solubility and freezing point lowering, corrosion, toxicity, relative cost or cost potential, effects on soils, plants, and water supplies, flammability, concrete compatibility, traction, friction, and highway performance. Information found in literature was supplemented or verified in the laboratory as needed.

Two candidate deicers—methanol and calcium magnesium acetate (CMA)—show promise as being as effective as NaCl. Methanol reacts almost immediately upon contact with snow and ice but is not as long lasting as NaCl and has greater utility as a fuel. CMA acts at about the same rate within the same temperature ranges and for approximately the same length of time as NaCl. CMA and NaCl produce similar changes in braking traction and skidding friction.

Unlike NaCl, CMA is not commercially available and a process for its manufacture would have to be developed. Unpurified CMA can be derived from solid wastes, primarily cellulose, which would aid traction and reduce production costs. In addition, limited laboratory tests indicate that CMA may be beneficial to most soils, unharmful to drinking supplies, and significantly less corrosive than NaCl.

The report is available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.

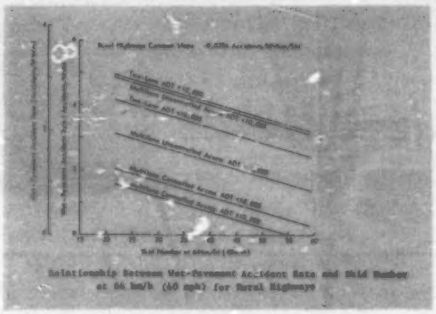


Equipment for Applying Epoxy Thermoplastic Paving Marking Material, Report No. FHWA-RD-79-130

by FHWA Materials Division

This report provides general specifications for an airless, low-pressure spray application system for epoxy thermoplastic pavement marking material. The main design and operation features of the equipment are documented. The specifications presented in this report are not intended to be rigid, but rather are guides for those seeking a system that is both simple and effective to use.

The report is available from the Materials Division, HRS-23, Federal Highway Administration, Washington, D.C. 20590.



Effectiveness of Alternative Skid Reduction Measures, Executive Summary and Volumes I–IV, Report Nos. FHWA-RD-79-21/25

by FHWA Environmental Division

These reports describe a study that examined the relationship between wet-pavement accidents and tire-pavement skid resistance, developed a computerized benefit-cost model for evaluating accident countermeasures, and produced a guide for improving pavement macrotexture.

Accident rate, skid number, and related data were collected for 2 years on 428 highway sections in 16 States. Extensive statistical analyses indicated a linear relationship between wet-pavement accident rate and skid number. The slope of the relationship (decrease in accident rate with increase in skid number) was independent of highway and area type and average daily traffic; however, the magnitude of the accident rate depended on such factors.

The executive summary presents the relationships between wet-pavement accident rates and skid numbers and includes a brief description of the study. Volume I describes the evaluation of accident rate-skid number relationships, and Volume II describes the development of a computerized benefit-cost model for wet-pavement accident countermeasures involving surface treatments as well as geometric, traffic control, and other remedial measures. Volume III contains instructions for using the computerized model, and Volume IV summarizes methods for measuring and achieving macrotexture.

Limited copies of the reports are available from the Environmental Division, HRS-43, Federal Highway Administration, Washington, D.C. 20590.

Implementation/User Items "how-to-do-it"

The following are brief descriptions of selected items that have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies.

U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division (HDV-20)
Washington, D.C. 20590

Items available from the Implementation Division can be obtained by including a self-addressed mailing label with the request.

User's Manual, Coal-Mine Refuse in Highway Embankments, Report No. FHWA-TS-80-213

by FHWA Implementation Division

Coal-mine refuse (CMR), the waste product of mining and processing coal for market, has commonly been disposed of in large refuse banks. These banks are not only unsightly but also pollute surrounding water and air.

This manual provides guidelines for using CMR in the design and construction of highway embankments. A separate section of

the manual presents data that are needed for a complete understanding of CMR properties, its origins, and regulations governing its disposal. Case histories of highway embankment construction with CMR are also included. The manual is intended for highway and environmental planners and those concerned with highway design, materials testing, and construction.

The manual may be purchased for $6 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00169-1).

Underground Disposal of Storm Water Runoff, Design Guidelines Manual, Report No. FHWA-TS-80-218

by FHWA Implementation Division

Disposal of storm water by infiltration into soil can provide a practical and attractive alternative to the more conventional and often more expensive storm water conveyance systems. This manual provides design guidelines for subsurface disposal of storm water by infiltration. The state of the art for design, maintenance, and construction of various infiltration drainage systems is presented, and infiltration basins, trenches, and wells are discussed in detail. Environmental and legal considerations relative to these systems are also presented.

Limited copies of the manual are available from the Implementation Division.

Hydraulic Flow Resistance Factors for Corrugated Metal Conduits, Report No. FHWA-TS-80-216

by FHWA Implementation Division

The resistance of the various corrugation shapes used in making corrugated metal conduits for highway drainage must be determined for reliable hydraulic design. The resistance varies widely for each of the corrugation shapes now available.

This report presents methods for estimating the resistance of all available corrugation shapes. Variables considered include conduit size and shape, corrugation shape, flow rate, flow depth, and manufacturing method. Resistance is presented in terms of the Darcy f or the Manning n, and design charts, geometric tables, and the International System of Units (SI) conversion factors are included. A method of estimating resistance for untested corrugations, along with design examples, also is included.

Limited copies of the report are available from the Implementation Division.

Development of Priority Accessible Networks—Provisions for the Elderly and Handicapped Pedestrians, Implementation Package 80-8

by FHWA Implementation Division

This manual describes a procedure for assessing the needs of elderly and handicapped pedestrians in urban areas. The procedure aids metropolitan planning organizations and public works agencies in identifying routes or corridors heavily traveled by the elderly and handicapped. These routes or corridors then can be surveyed to determine features that impede movement and need modification.

The manual is divided into two parts. Part one describes the planning procedure for developing an accessible network and part two describes specific problems experienced by elderly and handicapped pedestrians and recommends solutions.

The manual will be useful to persons in urban communities involved with pedestrian programs. The features shown in the manual can improve the safety and convenience of facilities for all pedestrians. Planners, designers, and safety program personnel in State and local agencies and citizen groups also may find this information useful.

The manual may be purchased for $6 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00163-2).

Development of the California Pavement Management System, Volume I, System Description, Report No. FHWA-CA-HM-7139-78-03

by California Department of Transportation

A pavement management system (PMS) has been developed to monitor deteriorating pavement, to disseminate repair strategy information, to substantiate cost-effective rehabilitation strategies, and to manage existing pavements in the State highway system. PMS begins with a Statewide

pavement condition field survey taken every 2 years. This survey identifies the location, nature, severity, and extent of pavement problems so data from the survey together with costs, rehabilitation strategies, traffic data, highway classification data, and other information can be computerized. The computer follows an engineering logic process to identify an appropriate level of repair for pavement. Because of the characteristic differences in flexible and rigid pavement problems and remedies, each pavement type is treated separately in the PMS. The end product is a series of reports describing pavement condition, rehabilitation strategy, and cost; candidate projects for various highway programs; and alternative repair strategies. This information will be used to establish the magnitude and character of the various highway programs, the priority of repair work, and the funding to provide a designated level of service for California.

Limited copies of the report are available from the Implementation Division.

Optimizing Maintenance Activities, Ninth Report, Rest Area Maintenance, Report No. FHWA-TS-80-210

by FHWA Implementation Division

This report summarizes the results of a value engineering study of rest area maintenance conducted by the States of Alabama, Illinois, Michigan, Virginia, and Wisconsin. It was estimated that more than $3 million annually could be saved among the five States with implementation of the recommendations of this study.

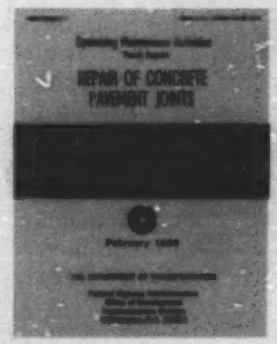
Because it was determined that 60 to 80 percent of rest area maintenance costs are for custodial labor,

reducing the number of labor hours will reduce the overall maintenance cost. Other recommendations concern sewage systems, trash and litter pickup, and contract maintenance.

Limited copies of the report are available from the Implementation Division.

Optimizing Maintenance Activities, Tenth Report, Repair of Concrete Pavement Joints, Report No. FHWA-TS-80-215

by FHWA Implementation Division



This report summarizes the results of a value engineering study conducted by the States of Iowa, Kansas, Nebraska, and South Dakota. The study covers repair of noncontinuously reinforced concrete pavements only. Deteriorated or defective joints in pavement cause unsatisfactory riding quality and performance of rigid pavements. When deterioration is so extensive that temporary surface patching no longer provides a safe, smooth riding surface, removal and replacement of a portion of the pavement are necessary.

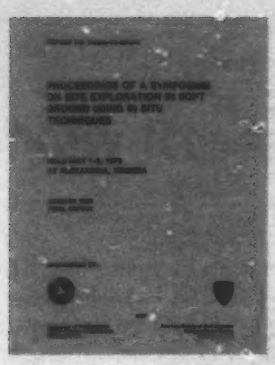
The joint repair problems of each State are individual and complex, requiring increased production in both contract and maintenance repair operations. Maximizing the use of machinery to accomplish the work instead of labor-oriented procedures is emphasized.

Limited copies of the report are available from the Implementation Division.

Proceedings of a Symposium on Site Exploration in Soft Ground Using In Situ Techniques, Report No. FHWA-TS-80-202

by FHWA Implementation Division

This report contains the proceedings of a 3-day symposium on site exploration in soft ground using in situ techniques held in May 1978. This symposium was the result of a research and development program to assimilate current information on the use and effectiveness of several new soft ground site exploration devices, including the pressuremeter (dilatometer), the vane shear, the borehole shear, and the static cone penetrometer.



The objective of the symposium was to review, summarize, and document any research conducted on these exploration devices and to present case histories in which each device was used to determine engineering soil properties and obtain design criteria. This research and development generated interest in the developmental stages and the potential uses of the many new site exploration techniques. The need for a summary of the state of the art of in situ site exploration testing of soils was identified.

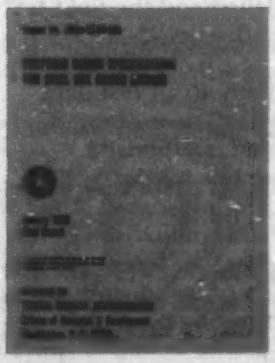
This report includes information on site exploration needs and compares exploration techniques as well as technical capabilities and uses of several site exploration devices. Classical pressuremeters are compared to the recently developed PAF (French) and Cambridge (English) self-boring pressuremeters. The report also includes a paper on the statics and dynamics of the standard penetration test (SPT).

The report may be purchased for $6.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00170-5).

Proposed Design Specifications for Steel Box Girder Bridges, Report No. FHWA-TS-80-205

by FHWA Implementation Division

In 1975, the Federal Highway Administration (FHWA), the American Society of Civil Engineers (ASCE), and the American Association of State Highway and Transportation Officials (AASHTO) recognized the need for a set of design specifications for long-span steel box girder bridges. Design specifications then in use were complex and unworkable. Therefore, FHWA sponsored the development of a set of practical design rules that would reflect the state of the art and that could be applied to steel box girders for long-span bridges. This report contains the first set of comprehensive but practical design rules for long-span steel box girder highway bridges. The rules also provide a potential basis for a uniform set of specifications for steel box girders of all span lengths. The report includes background information, the specifications, commentary on the specifications, and an annotated bibliography.

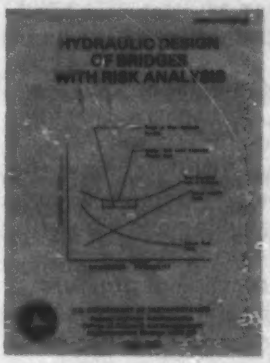


The proposed design specifications use the load factor design method and are presented in the standard AASHTO format. The specifications are now under review by the AASHTO Subcommittee on Bridges and Structures for possible inclusion in the 1980 AASHTO Interim Specifications.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 80 162324).

Hydraulic Design of Bridges With Risk Analysis, Report No. FHWA-TS-80-226

by FHWA Implementation Division



The hydraulic design of bridges involves an evaluation of the flood hazard to the highway and the effect of the proposed highway on lives, property, and stream stability. This manual presents example studies and reports that indicate that the total stream crossing, including the approach fill in the flood plains and all necessary waterway openings, should be designed and constructed for the least total expected cost to the public. The total expected cost to the public during the service life of

best copy available

the highway includes the capital investment in the highway, expected replacement and repair costs as a result of flood damages, expected user costs from traffic interruptions and detours, and expected backwater damages. Techniques for making engineering and economic studies for the least cost designs are presented with suggestions for managing the time and work required for these studies. A unique design flood is defined for each bridge as the flood whose upstream stage is equal to the lowest elevation of the approach fill or bridge deck.

Limited copies of the report are available from the Implementation Division.

French Self-Boring Pressure Meters (PAF 68 and PAF 72), Volumes I and II, Report No. FHWA-TS-80-209

by FHWA Implementation Division

The pressure meter is a soil exploration tool that determines the strength of soft soils inplace without undisturbed sampling and laboratory tests. The self-boring concept advanced the practice of pressure meter testing by eliminating the need for a predrilled borehole; the self-boring pressure meter probe drills its own precisely sized borehole with minimal disturbance to the surrounding soil.

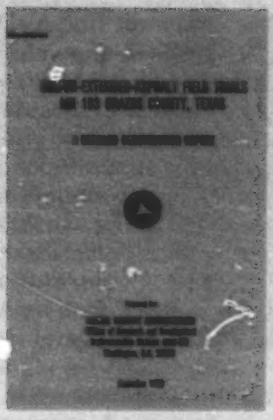
These reports include an evaluation of the PAF 68 and PAF 72 self-boring pressure meters and comprehensive instructions for running the pressure meter test. Although the self-boring technique shows potential for application in geotechnical engineering, it is considered a specialty technique and is still being evaluated.

Volume I, **Evaluation,** discusses the concept and practice of self-boring pressure meter tests. The evaluation project is presented in detail and the potential of the self-boring pressure meter as a soil exploration tool is reviewed. Applications of the self-boring pressure meter test to various geotechnical problems are discussed and recommended.

Volume II, **Instructions for Field Tests,** reviews preparations and procedures necessary to run a pressure meter test and to translate the resulting data into reliable soil parameters. The adaption of a probe-console system to different drill rigs is detailed and the field test procedures are discussed.

Of the three methods of test data reduction and analysis presented, a graphical method is proposed. Computer programs for data reduction are also presented. This volume includes a comparison of test results from several different probes and a discussion of soil disturbances during the self-boring pressure meter test.

Limited copies of the reports are available from the Implementation Division.



Sulphur-Extended-Asphalt Field Trials MH 153 Brazos County, Texas, Report No. FHWA-TS-80-214

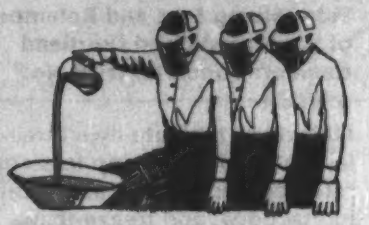
by FHWA Implementation Division

The sulphur in sulphur-extended-asphalt (SEA) binders replaces a part of the asphalt used in conventional asphalt paving and seems to enhance the strength and fatigue characteristics of the asphalt pavement mix. The increasing cost and potential shortage of asphalt have accelerated use of these binders.

This report contains the most comprehensive published description of equipment, materials, quality control procedures, and construction procedures used during the construction of pavement sections with SEA binders. The report also highlights design procedures and emissions control monitoring used before and during construction.

The report may be purchased for $3.75 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00165-9).

New Research in Progress



The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. Space limitation precludes publishing a complete list. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway or Transportation Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 2—Reduction of Traffic Congestion, and Improved Operational Efficiency

FCP Project 2P: Improved Utilization of Available Freeway Lanes

Title: Objectives and Parameters Associated With High Occupancy Vehicle Treatments. (FCP No. 42P3063)
Objective: Identify the objectives associated with each high occupancy vehicle treatment and determine how effectively the objectives will be met. Develop a manual to be used to analyze data to make recommendations for high occupancy vehicle applications.
Performing Organization: New Jersey Department of Transportation, Trenton, N.J. 08625
Expected Completion Date: December 1983
Estimated Cost: $98,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4C: Use of Waste as Material for Highways

Title: Materials Characterization of Recycled Bituminous Paving Mixtures. (FCP No. 34C4063)
Objective: Develop optimum mixture designs and evaluate their structural properties. Determine the effects of the environment, aging, and traffic. Evaluate road performance from field samples. Design alternative structural sections and predict performance using VESYS IIM. Develop interim guidelines for mixture and evaluate structural engineering properties.
Performing Organization: Resource International, Worthington, Ohio 43085
Expected Completion Date: January 1983
Estimated Cost: $134,000 (FHWA Administrative Contract)

FCP Project 4F: Develop More Significant and Rapid Test Procedures for Quality Assurance

Title: Correlation of Quality Control Criteria and Performance of Portland Cement Concrete Pavements. (FCP No. 34F3352)
Objective: Establish whether and how particular quality control tests, measurements, and observations relate to the future performance of portland cement concrete pavements and how much deficiencies indicated by particular test results will affect pavement life. Examine strength, density, air content, water/cement ratio, gradation, temperatures, steel placement, and joint construction.
Performing Organization: Resource International, Worthington, Ohio 43085
Expected Completion Date: January 1982
Estimated Cost: $152,000 (FHWA Administrative Contract)

FCP Project 4K: Cost Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments

Title: Long Term Rehabilitation of Salt-Contaminated Bridge Decks. (FCP No. 54K2092)
Objective: Develop and evaluate materials and procedures in conjunction with low-permeability overlays to prevent continued corrosion of reinforced steel surrounded by chloride-contaminated concrete bridge decks.
Performing Organization: Lehigh University, Bethlehem, Pa. 18015
Expected Completion Date: September 1982
Estimated Cost: $200,000 (NCHRP)

Title: Slump Loss and Retempering of Superplasticized (Portland Cement) Concrete. (FCP No. 44K2304)
Objective: Evaluate by a three-phase laboratory study slump loss and retempering, cement admixtures interactions, and freeze-thaw durability. Use different cements and admixtures. Try fly ash and use mini-slump tests, rheological tests, kinetics of reactions, adsorption, and microscopic tests.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Funding Agency: Illinois Department of Transportation
Expected Completion Date: October 1982
Estimated Cost: $80,000 (HP&R)

Title: Evaluation of the Construction and Initial Performance of Seven Polymer Concrete Overlays. (FCP No. 44K2324)
Objective: Construct under contract seven polymer concrete overlays (BNL method) on I-64 near Williamsburg, Va. The Virginia Research Council will do the testing and evaluation. Technical assistance will be given by the Implementation Division (HDV-22) and Brookhaven National Laboratory.
Performing Organization: Virginia Department of Highways and Transportation, Richmond, Va. 23219
Expected Completion Date: December 1985
Estimated Cost: $84,000 (HP&R)

Title: Corrosion Prevention of Reinforcing Steel in Concrete Exposed to Marine Environments. (FCP No. 44K3152)
Objective: Examine and correlate by a field and laboratory study the performance of systems containing different concrete mixtures, reinforcement, claddings, and coatings. Study voltage potentials in conjunction with cathodic protection, anodes, and conductive concretes. Measure chloride contents.
Performing Organization: Florida Department of Transportation, Tallahassee, Fla. 32304
Expected Completion Date: September 1983
Estimated Cost: $240,000 (HP&R)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Stress-Relieving Interlayers for Bituminous Pavements. (FCP No. 45D2664)
Objective: Install and evaluate the effectiveness of seven different reflective cracking SRI measures in six test sections. Examine two different crack types, crack densities, and overlay thicknesses.
Performing Organization: New York State Department of Transportation, Albany, N.Y. 12226
Expected Completion Date: March 1985
Estimated Cost: $112,000 (HP&R)

Title: Development of Design Procedures for ACHM Overlays on PCC Pavements. (FCP No. 45D2674)
Objective: Collect field data and materials testing data, analyze the data with mechanistic analysis, develop a procedure for designing against reflection cracking, and verify this with additional data.
Performing Organization: University of Arkansas, Fayetteville, Ark. 72701
Funding Agency: Arkansas State Highway and Transportation Department
Expected Completion Date: January 1982
Estimated Cost: $156,000 (HP&R)

FCP Project 5K: New Bridge Design Concepts

Title: Connections for Modular Precast Concrete Bridge Decks. (FCP No. 35K3022)
Objective: Develop new and improved connections for modular precast concrete bridge decks. Prepare recommendations for using these connections in the design and construction of highway bridges.
Performing Organization: Consulting Engineer Group, Glenview, Ill. 60025
Expected Completion Date: January 1982
Estimated Cost: $183,000 (FHWA Administrative Contract)

FCP Category 0—Other New Studies

Title: Develop a Maintenance Management System for Programing the Replacement of Reflective Pavement Markers. (FCP No. 40F0054)
Objective: Determine if the reflectivity of reflective pavement markers can be measured in the field on a production basis, if it is possible to establish a lower limit of reflectivity, and if it is possible, determine the lower limit. Design a program for replacing reflective pavement markers.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: June 1982
Estimated Cost: $190,000 (HP&R)

Instructions to Authors

All articles proposed for publication in *Public Roads* magazine are reviewed for suitability by the technical editors. Authors will be notified of acceptance or rejection as soon as possible.

Recent issues of the magazine should be reviewed for kind of articles, style, illustrations, tables, references, and footnotes. *Public Roads* follows the U.S. Government Printing Office Style Manual.

Submission of Manuscripts

Authors in the Washington, D.C., area should submit two copies of the manuscript to the managing editor.

Managing Editor (HDV-10)
Public Roads Magazine
U.S. Department of Transportation
Federal Highway Administration
Washington, D.C. 20590

Authors outside the Federal Government, or in State, city, or local government agencies, should submit two copies of the manuscript through appropriate Federal Highway Administration regional offices (see page 96).

Manuscript Treatment

Manuscripts should be typewritten, double spaced, with at least 1-inch margins on 8½- by 11-inch paper. Excluding art, 1 magazine page requires about 3 pages of manuscript. End each page with a completed paragraph. Type main headings flush left in initial caps. Subheadings should be flush left, and the first letter only capitalized. The article title and the name of each author should be typed on a separate page. If the article has been presented at a meeting, that should be indicated in a footnote at the bottom of the title page. Each page of the text should be numbered in the upper right margin. Because the Federal Government does not endorse products of manufacturers, avoid trademarks and brand names in articles unless their use is directly required by the objectives of the article.

Biography

A brief biography should be supplied. This should include the author's present position and responsibilities and previous positions relevant to the subject matter of the article. Biographies are limited to approximately 100 words. Biographies are accompanied by sketches of the authors drawn from photographs. Send a photograph of each author to the editor when the author is notified that the article has been accepted for publication.

Abstract

An abstract should be supplied with technical articles. For a *development paper*, the abstract should tell: (1) What has been accomplished, (2) its outstanding features, and (3) its applications, if known. The abstract of a *research paper* should focus on: (1) What has been accomplished, (2) its most important facts and implications, and (3) logical steps open to study.

Tables

Nonessential technical tables should not be included in the article. Each table should be typed on a separate page. It should be identified by an Arabic number and a caption. Note the location of the table in the text in the margin, but avoid putting it on the first page. Details of data already presented in tables or charts should not be repeated in the text.

Illustrations

Illustrations *referenced in the text* are called figures, and numbers and captions should be assigned to each. Organize the text so that illustrations can be scattered throughout the article. Avoid referencing several illustrations in one page to prevent problems in the layout. Black and white glossy photographs of good quality are preferred; however, color photographs and art are acceptable. Send original artwork to the editor when the author is notified that the article has been accepted for publication. Send legible copies with the manuscript. All captions (numbered and unnumbered) should be typed underlined on a separate page.

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Under present law and FHWA regulation, *Public Roads* is required to show the English equivalent in parentheses after every metric (SI) unit expressed in the text. In figures and tables, indicate equivalent units in a legend (for example, 1 km=0.6 mile); it is not necessary to show metric or English equivalents for each point plotted or quantity tabulated.

References

Number references consecutively in the body of the text, enclosed in parentheses and underlined. Copyrighted material referenced and quoted will require copyright releases. Unpublished material referenced in the text will be described in a footnote. Type citations on a separate page under the heading REFERENCES. Number citations in the same manner as in the text and list in the same sequence.

Galley Proofs

Galley proofs will be sent to authors for their inspection.

For further information, contact the editor: (703) 557-4304.

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THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

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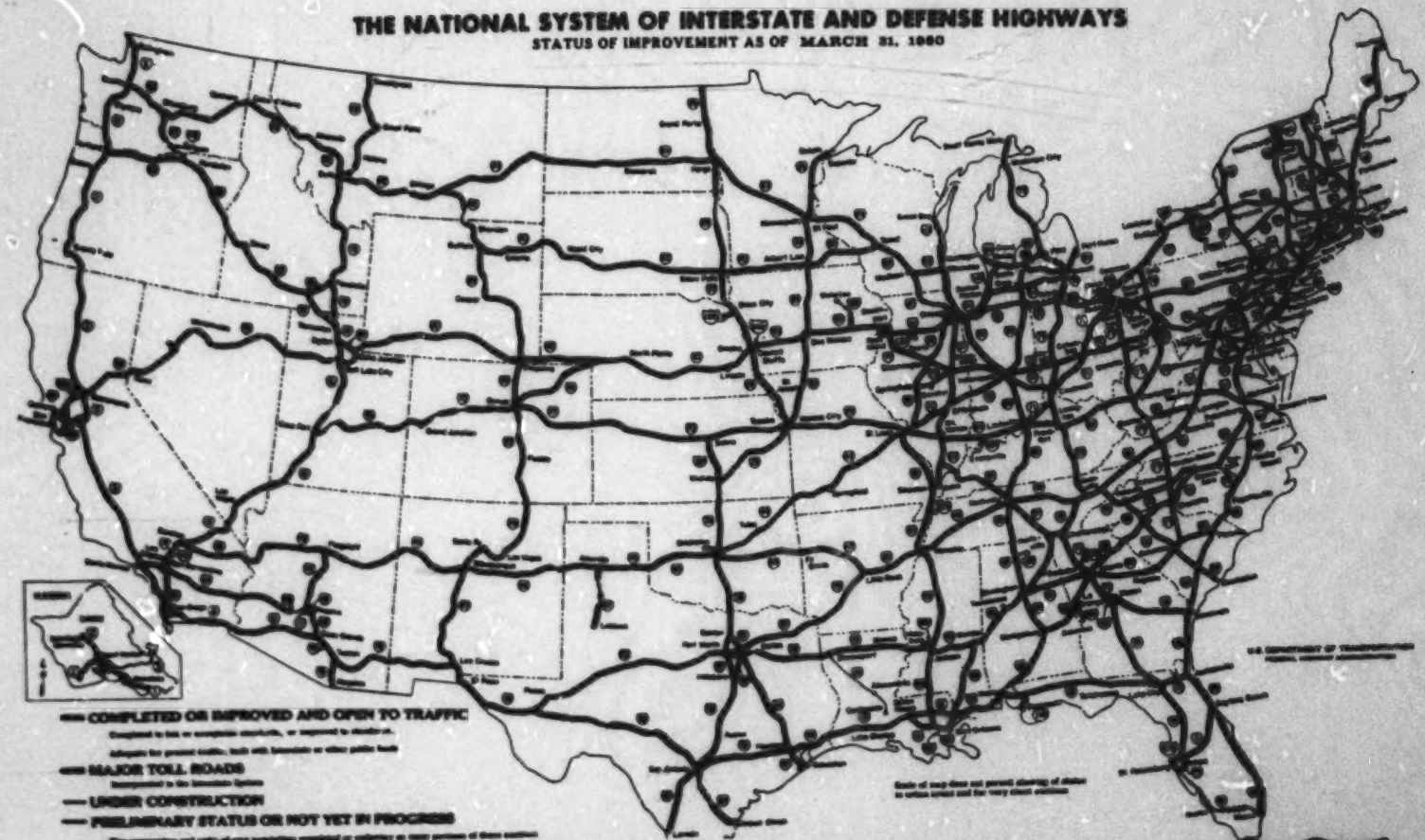
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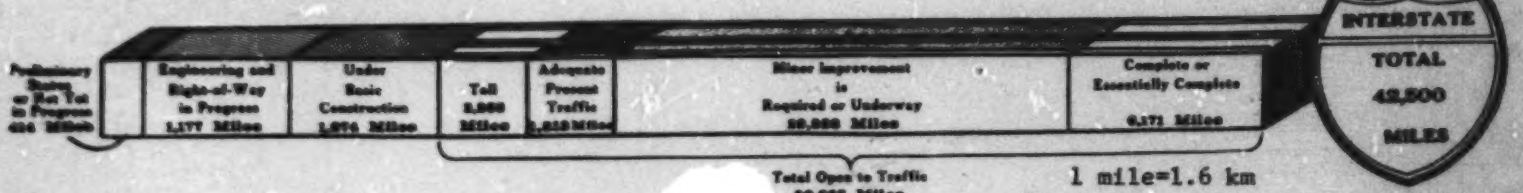
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Adequate Present Traffic 1,818 Miles
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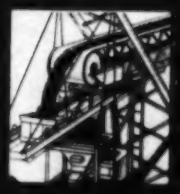
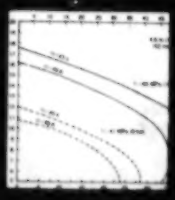


Process Control for Aggregate Production and Use

Bridge Movements and Their Effects



Applications of High Strength Concrete for Highway Bridges

public roads

A JOURNAL OF HIGHWAY RESEARCH AND DEVELOPMENT